CONNECTICUT RIVER FLOOD CONTROL

BIRCH HILL DAM

MILLERS RIVER
MASSACHUSETTS

ANALYSIS OF DESIGN

1940

CORPS OF ENGINEERS, U. S. ARMY

U. S. ENGINEER OFFICE PROVIDENCE, R. I.

CONNECTICUT RIVER FLOOD CONTROL
ANALYSIS OF DESIGN

BIRCH HILL DAM
MILLERS RIVER
MASSACHUSETTS

JANUARY 1940

CORPS OF ENGINEERS, U. S. ARMY

U. S. ENGINEER OFFICE

PROVIDENCE, RHODE ISLAND

BIRCH HILL DAM

ANALYSIS OF DESIGN

CONTENTS

PERTINENT DATA

Drainage area, square miles

Net (excluding Lower Naukeag)	155
Gross (including Lower Naukeag)	175

Elevations

Top of dam	864
Spillway crest	852
Pool at maximum flood	859
Low water in stream at dam site	810
Streambed at dam site	808
Top of temporary cofferdam, upstream	820
Top of upstream cofferdam	832
Top of downstream cofferdam	823

Hydraulic data

Reservoir capacity

Elevation	Area of lake in acres	Capacity in acre feet	Corresponding storage in inches of run-off from the gross drainage area
Below El. 852, spillway crest	3,200	49,900	5.3
Total below El. 864, top of dam	4,630	99,000	10.5
Total below El. 859 (surcharge for spillway design discharge)	4,000	76,000	8.1

Years of hydrographs	1916 to 1937, inclusive
Maximum flow, March, 1938 peak	16,000 c.f.s.
Maximum flow, 24 hour mean	11,650 c.f.s.
Estimated peak inflow, spillway design flood	51,600 c.f.s.
Estimated peak outflow, spillway design flood	56,600 c.f.s.

Estimated volume, spillway design flood	20.66 inches
Estimated volume, outlet design flood	9.15 inches
Estimated peak inflow, outlet design flood	17,600 c.f.s.
Diversion during construction (Pool at El. 830.0)	3,500 c.f.s.
Approximate tailwater elevation at maximum flood (Pool at El. 859)	841.3 ft.
Approximate tailwater elevation outlet discharge 10,500 (Pool at El. 852)	828.0 ft.
<u>Outlet capacity</u>	
Pool at El. 852	10,500 c.f.s.
<u>Spillway capacity</u>	
Pool at El. 864 (Top of dam)	124,000 c.f.s.
Pool at El. 859 (Maximum flood elevation)	56,600 c.f.s.
<u>Embankment</u>	
Maximum height above streambed	56 ft.
Total length on top	1,400 ft.
Top width of dam	25 ft.
Total fill in embankment, including blanket	282,000 cu.yd.
Total impervious core blanket and cut-off trench	65,000 " "
Total random fill	74,000 " "
Total pervious fill	78,000 " "
Total rock fill and dumped riprap	41,000 " "
Total gravel bedding	3,000 " "
<u>Outlet works</u>	
Shape outlet channel	open cut, trapezoidal
Total common excavation, outlet works	128,000 cu.yd.
Total rock excavation, outlet works	54,000 " "

Length of intake channel	1,500 ± ft.
Length of outlet channel	1,150 ± ft.
Bottom width of intake channel in earth	70 ft.
Bottom width of intake channel in rock	40 ft.
Sill elevation gate structure	815
Size of gates	6 x 12
Number of gates	4

<u>Spillway</u>

Length of spillway crest, concrete	1,100 ± ft.
Total common excavation, spillway	36,000 cu.yd.
Total rock excavation, spillway	4,000 " "

<u>Operating house</u>

Size 23'-9" x 49'-9" x (height) 28'

Structural steel frame, brick walls

Individual electric powered cable hoists

Overhead crane capacity 15 tons

25 KW. gasoline-electric standby unit

<u>Concrete quantities</u>

Total	14,800 cu.yd.
Operating house substructure	1,400 " "
Spillway weirs	1[illegible],[illegible]00 " "
Retaining walls and channel lining	1,030 " "
Access road bridge and abutments	170 " "

I. INTRODUCTION

I. INTRODUCTION

A. AUTHORIZATION AND PAST REPORTS. - The Birch Hill Dam is authorized by the Flood Control Act approved June 22, 1936 (Public No, 738, 74th Congress), as amended by Public No. 111, 75th Congress, approved May 25, 1937 and further amended by Public No. 761, 75th Congress approved June 28, 1938. The original project report was submitted December 8, 1936. This dam is also one of twenty dams included in the comprehensive system of flood control reservoirs recommended by the District Engineer in "Report on Survey and Comprehensive Plan for Flood Control in the Connecticut River Valley" dated March 20, 1937, approved by the Chief of Engineers, November 29, 1937 and published as House Document No. 455, 75th Congress, 2d Session and approved by Congress in the Flood Control Act of June 28, 1938.

B. EFFECT OF BIRCH HILL RESERVOIR IN REDUCING FLOODS. - The Birch Hill Reservoir has a flood control capacity of 50,000 acre-feet, which is equivalent to 5.3 inches of run-off from its gross drainage area of 175 square miles. The proposed Lower Naukeag Reservoir would control 20 square miles of this area, thus leaving a net drainage area of 155 square miles to be controlled by the Birch Hill Reservoir. The capacity of the latter corresponds to 6 inches of run-off of this net area of 155 square miles. The Birch Hill Reservoir will provide a considerable degree of flood protection for Athol, Massachusetts and the several communities on the Millers River below. It will also have considerable value for the reduction of floods on the Connecticut River below the mouth of the Millers River. As a part of the comprehensive plan of reservoirs for the Connecticut

River flood control, it will reduce flood discharges on the average at various Connecticut River points as follows:

Locality	Drainage Area (square miles)	% Reduction of Peak discharge
Montague City	7,840	2.8
Holyoke	8,284	2.8
Springfield	9,587	2.2
Thompsonville	9,637	2.2
Hartford	10,643	2.0

This reservoir, considered separately, would have an important flood-reducing effect during many Connecticut River floods. Under certain circumstances when the run-off from the Millers River should not happen to combine in substantial synchronism with the highest flood discharge on the Connecticut River, the flood-reducing effect of this reservoir in the main valley would be minimized. The reservoir outlet and gates are designed with sufficient discharge capacity to permit flexibility of operation for best flood storage in reducing any type of flood that may occur.

C. <u>CONSERVATION STORAGE</u>. - No provisions have been made for conservation storage.

D. <u>GENERAL BRIEF DESCRIPTION OF THE DAM</u>. - The dam will be an earth structure having a maximum height of 56 feet from the river bed to the top of the dam and a total length of approximately 1400 feet. The embankment will be placed by the rolled-fill method. Both the upstream and the downstream slope will be riprapped with dumped rock.

The reservoir outlet will consist of a gate-controlled open-cut channel excavated in ledge rock with concrete training walls in the right abutment and discharging into the river well below the toe of the dam. The gates will be housed in a concrete gate structure located adjacent to the center line of the dam. Access to the Birch Hill Dam will be provided from South Royalston on an access road constructed on the abandoned railroad foundation and the top of the dam. The main spillway will be constructed on rock in a natural saddle on the right bank about 1000 feet downstream from the center line of the dam. A small auxiliary concrete spillway weir will be located in a saddle adjacent to the main spillway. The spillway flow will be carried in a natural depression and will discharge into the Millers River below the toe of the dam. For a general plan of the dam and appurtenant structures, see Plate No. 1.

II. SELECTION OF SITE

II. SELECTION OF SITE.

A. GENERAL LOCATION. - The Birch Hill Dam site is located on the Millers River, Massachusetts, a tributary of the Connecticut River which has a total drainage area of 390 square miles. The gross area that will be controlled by the reservoir, including the area tributary to the Lower Naukeag reservoir is 175 square miles. The site selected is 1.3 miles northeast of South Royalston, Worcester County, Massachusetts, about 27.3 miles above the confluence of the Millers and Connecticut Rivers.

B. SITES CONSIDERED. - A comprehensive plan of a series of smaller reservoirs was investigated to avoid fairly large local damage at Baldwinsville, Massachusetts. This plan included Hydeville and Priest Pond reservoirs which were submitted as alternate sites in the Comprehensive Plan, House Document 455, and a reservoir on the Otter River, a tributary to the Millers River near Baldwinsville. Several potential sites on this river were investigated. This combined system was found to be less effective than the Birch Hill Reservoir. From the consideration of reservoir capacity, geologic conditions and construction costs, the selected site is considered the most feasible site on the Millers River above Athol, Massachusetts.

C. SELECTION OF SITE. - The selection of a dam site on the Millers River was made with flood protection of the City of Athol as a primary consideration. Upon elimination of the system of three smaller reservoirs above South Royalston, three potential sites for the Birch Hill Dam were investigated. A thorough investigation of the river valley above and below South Royalston, including map studies field reconnaissance, reservoir and site surveys, geological investigations and comparative studies of railroad relocation costs, led to the selection of the site. The dam site as included in House Document 455 and a site within South Royalston compare favorably in construction costs with the one selected; however, the cost of railroad relocation for those sites would be far in excess of that required for the seclected site. The selected site is satisfactory from the viewpoint of economy, safety, and utility, and was found adaptable to a design of a safe dam and appurtenant structures, the cost of which have a favorable ratio to the benefits derived. See Plate No. 3.

D. SELECTION OF TYPE OF DAM. - A study of the geologic conditions and the location of bed rock resulted in the adoption of an earth-fill dam as being the most practicable type for the site. Although rock is close to the surface on both banks, a deep deposit of overburden occurs in the river valley and the cost of a concrete structure would be prohibitive. A rock-fill dam would have been physically feasible, but the cost was found to be excessive. The materials available for earth fill at or near the dam site is better suited for hydraulic fill than for rolled fill construction; however, owing to the low height of the dam, extensive use of structure excavation and small quantities required, the cost of hydraulic fill would be excessive. The presence of the railroad in the river valley at the left bank and the river near the right bank provides a work area on which suitable structure excavation can be deposited without rehandling. Hydraulic construction would require diversion of the river through the outlet channel and removal of the railroad line before embankment operations were begun. All of the rock and suitable structure excavation, supplemented by borrow excavation, will be used for the embankment.

III. GEOLOGIC INVESTIGATIONS.

III. GEOLOGICAL INVESTIGATIONS

A. NATURE OF VALLEY. - The reservoir will be formed in the main valley of the Millers River and its tributary valleys of the Otter River and Priest Brook. The stretch of the Millers River valley westerly of South Royalston, located about 1 mile downstream of the site, is sharply defined. The valley through here is comparatively narrow and steep-sided. The stream obviously is flowing in its ancient preglacial valley although considerably above its buried rock floor.

Easterly of South Royalston the valley is less sharply defined. About 2 miles upstream of the site the valley bottom becomes much wider extending northerly in two divergent valleys, now occupied by the Millers River and Priest Brook, and southerly and southeasterly in the valley occupied by Otter River. Thick glacial outwash deposits of sand and gravel occur throughout these tributary valleys and deeply bury the bedrock. In the higher elevations the underlying rock foundations are obscured by glacial till deposits. Here the glacial overburden is relatively thin and bedrock is exposed at many points.

B. METHOD AND EXTENT OF EXPLORATIONS. - Subsurface explorations were accomplished by numerous core borings, auger borings, and test pits. These investigations were made (1) to locate and sample rock beneath both abutments, with special reference to location of the discharge channel, (2) to locate and sample rock at both spillway locations and stream control works, (3) to determine the character, thickness, and quality of overburden in both abutments, (4) to determine the extent of cohesionless fine sand deposits beneath the valley flood plain and to obtain undisturbed samples thereof, (5) to investigate the thickness and extent of organic deposits in the valley flood plain, and (6) to investigate potential borrow areas. The location of these explorations is shown on Plates Nos. 4 and 7, and the records of each on Plates Nos. 5, 6 and 7.

1. Core Borings. - All overburden drilling was done by drive sample borings. Frequent soil samples of 1-1/2-inch diameter were obtained in solid and split sampling spoons. In addition, two bore holes located in the middle of the valley were placed, samples being obtained within 2-inch brass liner tubes. Two 6-inch diameter holes were also completed to provide a maximum quantity of cohesionless fine sand for triaxial shear tests. Precautions were taken at all times to assure that samples were undisturbed by wash water. Rock samples were obtained as rock cores from many of the borings. Penetration of bedrock by core drilling ranged from 10.0 feet to 46.0 feet, the average penetration being 20.1 feet. All drilling was done by Government-owned equipment and hired labor.

Particular attention was paid throughout the investigations to sampling methods and procedure. Numerous soil cores were obtained where the overburden had sufficient apparent cohesion to hold the soil grains together. Where such a condition was encountered the sample was obtained in as undisturbed condition as possible and immediately coated with paraffin. Additional relatively undisturbed samples were obtained in brass liners.

Sampling of cohesionless fine sand in the foundation is being continued. For these additional samples, a new type of spoon and method will be used. The chief difficulty in obtaining undisturbed samples in materials of this type below the water table is that, upon retracting the sampling spoon from the depth sampled, the sample is often dislodged from the spoon and much or all of it is lost. In this new method it is planned to form a grouted plug at the bottom of the sample by introducing chemicals (sodium silicate and calcium chloride) which underground will react sufficiently to form a binder and hold the sample in a natural undisturbed condition. Sampling investigations by this method are now being made, but it is not possible at this time to give any information as to the results obtained.

2. Auger borings. - Auger borings were used throughout the valley bottom to outline the extent and thickness of organic deposits known to occur in portions of the flood plain.

3. Test pits. - Numerous shallow test pits, ranging in depth from about 2.0 feet to about 12.0 feet, were excavated in the overburden of both abutments, and in borrow areas. Undisturbed box samples for permeability tests were obtained from those pits in the abutments. In addition known volume samples were obtained for natural density determinations, as well as bag samples for laboratory compaction tests.

4. Shovel cuts. - Two large open cuts will be excavated by power shovels, one located at the site of the gate house structure, the other in the inlet channel. These cuts will be made to provide a better means for inspecting the overburden and underlying rock, and provide prospective bidders with adequately large excavations for better appraisal of excavation difficulties.

5. Other investigations. - Reconnaissance explorations by the

seismic method were made at the dam and several alternative spillway sites. A strip about 400 feet wide and about 3 miles long located on the south side of the river, was similarly investigated for the required railroad relocation. No other important geological investigations were made. Information obtained from all the explorations satisfactorily discloses the geological conditions at the site. Judging from reconnaissance of the reservoir area and general studies of the surrounding region, there are no geologic problems of consequence involved in the reservoir area above the dam site.

C. SITE. - Bedrock is not exposed in either abutment, but is obscured by an overburden composed of glacial sediments. In the extreme right abutment, bedrock is located at a depth of about 20 feet, beneath an impervious overburden comprised largely of Class 7 material, which in turn lies beneath an overlay made up of Classes 4, 5 and 6 approximately 5 feet thick. The rock surface dips steeply towards the south, and at the river is located at a depth of about 100 feet beneath the stream. Bedrock in the left abutment is located at an average depth of about 20 feet.

The Class 7 materials mentioned above comprises a compact glacial till formation which is extensively developed throughout both abutment areas. They form a relatively impervious formation, to which the impervious section of the dam will be bonded. Large boulders are scattered on the surface, and in the overburden. Those on or near the surface are of granitic character and essentially unweathered compared to those buried at greater depth in the overburden. Several of the bore holes disclose a very compact glacial till immediately above bedrock. This till often contains a large percentage of decayed rock, in some cases derived from disintegrated boulders and cobbles. In other cases material of similar appearance and compactness is judged to be decayed and disintegrated bedrock. Where such conditions were found, it was not possible to locate the exact line of demarcation between overburden and decayed bedrock. Whichever interpretation is correct, action of the sampling spoon and drill indicates the materials upon excavation will behave similar to a compact overburden.

Foundation materials beneath the flat valley bottom occur in three major strata. The upper stratum, approximately 6 to 12 feet thick is made up of irregular deposits of sand and gravel (Classes 2, 3, 4, 5, 7). An organic deposit ranging in thickness from about 1 foot to about 4 feet occurs at the surface on the south side of the valley near the railroad embankment. A layer of loose cohesionless fine and medium sand (Classes 4 and 6), approximately 60 feet thick in the middle of the valley, occurs beneath the uppermost stratum. Beneath this intermediate layer occur well compacted variable deposits, Classes 7 and 9. Investigations have not revealed the occurrence in any great amount of either clay or glacial silt.

Bedrock is composed of two types (1) medium to coarse foliated gray granite and (2) biotite mica schist. These two types occur in both a simple and complex relation. In its simplest relation the granite occurs in large intrusive masses and the schist in inclined beds or bands. In their most complex relation the rocks vary from true schists, derived by metamorphism of sedimentary strata, to gneissoid granites, clearly of igneous origin. Traces of material, more or less clearly defined, and resembling the original schists are nearly always associated with the granitic masses. Between the two extremes of granite and schist there are gradations, designated as gneiss, which appear to be mixtures of these two types. The granite is broken and fractured near the surface, and in some areas is slightly weathered. The schist is deeply weathered and disintegrated. The extent of this condition is dependent upon the amount of injected igneous material present in the schist.

D. NATURE OF EXCAVATIONS. - Excavation of the stream control channel will be a major construction operation. The inlet channel, near the river, will be excavated in sand and gravel (Classes 5 and 7), and the outlet channel in medium and fine sand (Classes 4 and 6). In the higher portions of the hill forming the abutment on the east, excavations will be made in a compact bouldery glacial till made up largely of Classes 7 and 9. On the west slope of the hill the overburden is made up of fine and medium sand of the type found in the outlet channel.

For a distance of approximately 1,000 feet at bottom grade, the excavation will be in rock. The rock cut will have a maximum depth of about 40 feet at station 16+00. From the upstream end to a point immediately downstream of the gate structure, much of the rock is weathered and fractured, but that in the downstream portion of the cut is relatively more sound. About 152 linear feet of the channel will be provided with concrete lining. The ultimate extent of lining will be decided in the field.

The comparatively shallow excavations required for the foundations of the concrete spillway structure will be made in sand and gravel (Classes 5 and 7). Excavations of the key trench, for bonding impervious core section, to abutment and foundation, will also be made in sand and gravel including Classes 2, 4, 6, 5, and 7.

E. DISCHARGE CHANNEL AND SPILLWAY. - The principal features pertaining to the overburden and rock at these structures is covered in the preceding paragraphs in Sections III C and D.

F. SUBSURFACE LEAKAGE. - Ground water observations indicate a water table closely parallel to the ground surface. This highwater table, together with the glacial character of materials known to occur, indicates both abutments to be relatively impervious. By properly bonding the core section to the impervious abutments, very little leakage through these sections is anticipated. Most of the leakage through the foundation will be concentrated in the comparatively thin strata lying between the bottom of the cut-off trench and the top of the thick stratum of fine and medium sand. The seepage path of this leakage beneath the dam will be lengthened by construction of an upstream blanket formed of spoiled overburden.

IV. FLOOD CONTROL HYDRAULICS

IV. FLOOD CONTROL HYDRAULICS

A. HYDROLOGY.

1. Rainfall records. - In the watershed above the Birch Hill site and in the surrounding area, there are 39 rainfall stations, with periods of record ranging from 8 to 75 years. Their locations are shown on Plate No. 9 and their periods of record on Plate No. 10. Pertinent general precipitation data are:

Mean annual rainfall	- 43.61 inches (including snow).
Maximum annual rainfall	- 57.82 inches (including snow).
Minimum annual rainfall	- 32.00 inches (including snow).
Mean annual snowfall	- 58.8 inches.
Approximate water equivalent of mean annual snowfall	- 6.0 inches.

Mean Monthly Rainfall

January	3.53 in.	April	3.55 in.	July	4.02 in.	October	3.28 in.
February	3.13 in.	May	3.32 in.	August	4.03 in.	November	3.71 in.
March	3.65 in.	June	3.87 in.	September	4.15 in.	December	3.37 in.

Maximum storm of record in this general region:

Date - September 17-21, 1938.

Duration - 98 hours.

Maximum average depth of rainfall on 175 square miles - 15.7 inches.

a. Maximum predicted storm. - The maximum predicted storm on the Birch Hill Watershed was selected as having a volume and distribution as recently determined by the U. S. Weather Bureau in collaboration with the Office of the Chief of Engineers, and based upon a recent study of rainfall in New England. The greatest storm of record in the Connecticut River Watershed occurred in September 1938. This storm had a duration of 98 hours and a maximum average depth of 15.7 inches of rainfall on an area of 175 square miles. The area-depth graphs and the locations of the centers of this storm and the seven other greatest summer and fall storms of record in the northeastern United States are shown on Plate No. 11. Similar graphs for the five greatest winter and spring storms of record are shown on Plate No. 12. All storms studied were listed as 1-1/2 to 3-day storms. The selected maximum predicted storm for the Birch Hill drainage area of 175 square miles, as shown on Plate No. 13, has a storm depth of 16.5 inches and a duration of 24 hours. The maximum storm depth for summer and fall floods of record in the northeastern United States, as shown on Plate No. 11, is 15.7 inches, and for winter and spring storms, as shown on Plate No. 12, is 6.0 inches.

b. Time - intensity distribution. - The distribution of the maximum predicted storm in a period of 24 hours was determined from the same study by the U. S. Weather Bureau, shown on Plate No. 13. For a drainage area of 175 square miles the curve of intensity versus duration has been drawn as shown on Plate No. 14. For means of comparison the curves of maximum rainfall intensities as determined by David L. Yarnell in the U. S. Department of Agriculture Miscellaneous Publication No. 204 for a frequency of 100 years are shown on this plate. A curve of maximum intensities for a frequency of 100 years for Boston, Massachusetts, as given by Charles W. Sherman in his paper, "Frequency and Intensity of Excessive Rainfall in Boston, Massachusetts," Transactions of the American Society of Civil Engineers, Volume 95, 1931, is shown also. This latter curve is based on 50 years of recording gage records and is included because it is the result of a very thorough study of excellent records. Its value in the present study is to indicate the probable variations of intensity with time. The time-intensity curves have been plotted on logarithmic paper in order to provide a better comparison between the results of these three studies. It will be noted that the adopted maximum curve has the same general shape as those given by Yarnell and Sherman but with much higher values of rainfall. Three points for the September 1938 storm, determined from actual non-recording gage records, are also shown on Plate No. 14. For shorter periods of time, the September 1938 storm intensity falls well below any of the curves shown. For use in synthesizing the computed spillway flood, the storm intensity was assumed to be nearly symmetrical about a vertical time axis as shown on Plate No. 18. This distribution was found to be fairly true for storms of record and produces the maximum flood obtainable with any distribution.

2. Run-off records. - Run-off records of the Millers River Watershed have been collected by the U. S. Geological Survey for the past 24 years. The locations of the gaging stations are shown on Plate No. 9 and their periods of record on Plate No. 15. The hydrographs for the Birch Hill drainage area as shown on Plate 16 was obtained by applying a factor to the records of the U. S. Geological Survey Station on the Millers River near Winchendon, Massachusetts, (drainage area 83.9 square miles) from 1916 to date. This factor is a variable depending on the discharge, and is based on discharge records secured by this office at the dam site and a relation of peak discharges of the March 1936 and September 1938 floods furnished by the U. S. Geological Survey for various points along the Millers River.

Pertinent run-off data are:

Mean annual run-off	22.90 inches
Maximum annual run-off	32.29 inches
Minimum annual run-off	10.94 inches

Note: These figures do not include runoff since Sept. 30, 1937.

Mean monthly run-off and percent of run-off

January	1.93 inches	54.7%	July	1.08 inches	26.9%
February	1.51 inches	48.2%	August	.84 inches	20.8%
March	3.90 inches	107.0%*	September	.83 inches	20.0%
April	4.75 inches	134.0%*	October	.85 inches	25.9%
May	2.48 inches	74.8%*	November	1.34 inches	36.1%
June	1.57 inches	40.5%	December	1.82 inches	54.0%

* Includes run-off from melting snow.

Through field investigation and research, historical knowledge of several of the most severe floods on the Millers River within the past 150 years was obtained. They are known to have occurred in September 1938, March 1936, November 1927, February 1900, March 1896, April 1895, July 1887, December 1878, October 1869, April 1862, May 1854, March 1846, March 1843, January 1841, February 1824, and March 1801. Prior to June 1916, the exact order of magnitude is not known. Since then, the September 1938 flood is definitely known to have been the highest. From a study of peak discharges of the September 1938 flood at several locations on the Millers River, and from a composite flood hydrograph at Birch Hill dam site built up from component areas for which the flood hydrographs were available from records or were reconstituted by use of unit graphs based on the Connecticut River empirical relations, the September 1938 peak discharge at the Birch Hill dam site is estimated to have been 16,000 cubic feet per second.

a. Frequency of floods. - The probable frequencies of peak discharge at the Birch Hill dam site are shown on Plate No. 17 for the following seasonal periods:

January and February	May through October
March and April	November and December

These curves serve only as an approximate index of what may be expected in the future and are useful principally as a guide for planning flood protection and stream diversion during construction and for determination of the capacities of operating features of the dam.

b. Maximum predicted floods. - The computed spillway flood was determined by applying the unit hydrograph (described hereinafter) derived from the composite September 1938 flood, to the maximum predicted storm, assuming a uniform infiltration rate of .05 inch per hour. On this basis, the percent of run-off during the most intense 3-hour rainfall period would be 98 percent, and during the entire storm period, 92.7 percent. The resulting flood hydrograph is shown on Plate No. 18. Its volume is 15.3 inches and its peak discharge is 38,200 cubic feet per second, or 217 cubic feet per square mile, over twice as large as the maximum flood within historical knowledge, that of September 1938. An envelope curve of summer and fall flood peak discharges for New England is shown on Plate No. 19. The maximum predicted flood at the Birch Hill dam site exceeds the value of 121 cubic feet per second per square mile for a drainage area of 175 square miles, taken from the envelope curve for the Connecticut River tributaries east of the main stem and south of the Ammonoosuc River by 80 percent. An envelope curve of winter and spring flood peak discharges for New England is shown on Plate No. 20. The maximum predicted flood as herein determined would result from the worst possible storm magnitude, intensity distribution, rate of infiltration and watershed run-off conditions.

c. Unit hydrographs. - Two general methods were used to derive unit graphs for the purpose of determining the computed spillway floods.

(1) Derived from September 1938 flood. - One of these is by derivation from the maximum flood of record, that of September 1938, at the Birch Hill dam site. The rainfall excess for this storm fell in a concentrated period of 42 hours. The composite hydrograph used for this derivation was built up by numerically adding hydrographs of component areas based on the available hydrographs of record, and assumed hydrographs derived by application of a straight drainage area factor to the hydrographs of record. This composite hydrograph has a peak discharge of 17,900 cubic feet per second as shown on Plate No. 21. It is known to be more severe than the actual flood of occurrence because no account has been taken of the reduction in component hydrographs which would occur when routed through the main stem valley storage. This peak discharge is also much higher than an envelope of other peak discharges for the September 1938 flood on the Millers River. Also, for other locations in the Millers River Watershed, the September 1938 flood results in a more severe unit graph than any other flood of record. Consequently, the derived graph, shown on Plate No. 22, should be the most critical for use in computing maximum predicted floods.

(2) Derived from empirical relations. - The other method of obtaining a unit graph is by determination of critical dimensions of the unit graph from empirical relations derived from topographic features of the watershed. These empirical relations were defined by a

thorough analysis of the unit hydrographs and watershed topography at 22 gaging stations in the Connecticut Basin. G. T. McCarthy determined that the shape of the unit hydrograph for any watershed is principally a function of its drainage area, stem pattern, and the slope of a graph of area versus elevation equalled or exceeded ("The Unit Hydrograph and Flood Routing," presented at the conference of the North Atlantic Division, U. S. Engineer Department, held at New London, Connecticut, June 24, 1938). From the empirical relations, the unit hydrograph resulting from a twelve-hour storm of constant intensity is fixed in shape by five values; namely,

Peak rate of discharge

Rate of discharge 12 hours after the peak

Time from beginning of storm to peak discharge

Total duration of unit hydrograph

Total volume of run-off under the unit hydrograph

These values and the unit hydrograph they define for the Birch Hill Watershed are shown on Plate No. 23. The unit graph derived from the empirical relations has a higher peak discharge than the one derived from the 1938 flood. The reason for this is that the Millers River and its tributaries are very slow in run-off characteristics and yield smaller peak flows than other watersheds in the Connecticut Basin. This fact was first apparent when the original studies for establishing the empirical unit graph relations were being made; the Millers River values fell far out of line with other data. Subsequent use of the empirical relations for various watersheds has shown them to be best adjusted to flashy watersheds, and to yield peak discharges which are too high and concentration times which are too brief for the very slow watersheds. The difference between the two unit hydrographs developed would be even greater had the September 1938 unit hydrograph been derived from a recorded hydrograph at the Birch Hill dam site rather than from the more severe composite hydrograph used.

(3) Short period unit hydrographs. - In order to reflect accurately the maximum instantaneous rate of peak discharge resulting from a storm with rapidly changing intensities of precipitation, unit hydrographs having much smaller unit periods of rainfall than twelve hours were used. As shown on Plate No. 22, the unit graph derived from the September 1938 storm was made for 6-hour rainfall periods, and broken down into 3-hour unit graphs. In the case of the empirical relations, the original derivation is for a 12-hour unit rain, and unit graphs for 6 and 3 hours are derived from it, as shown on Plate No. 23.

(4) Selected unit graph. - The unit graph derived from the composite September 1938 flood was adopted for use, because the rainfall and run-off conditions of the September 1938 flood were very similar to those established for computation of the maximum predicted flood; namely, the rainfall excess of 7.5 inches occurred in 42 hours (as against 24 hours for the computed flood) and had a distribution comparable to that selected for the computed flood. The rainfall excess of the September 1938 storm was immediately preceded by 1.3 inches of rain in the previous 12 hours and 3.0 inches during the 2-1/2 previous days, none of which rain appears in the actual hydrograph. Thus, it is probable that during the September 1938 flood the ground was saturated and infiltration occurred at a fairly uniform rate, similar to that assumed for the maximum predicted flood.

d. Tailwater rating curve. - The tailwater rating curve for Birch Hill dam site is shown on Plate No. 24. It was developed in its lower range by backwater computations from the low dam above South Royalston, Massachusetts, which is the control. In its higher range, it was determined by backwater computations from downstream control sections.

3. Maximum predicted wave heights and freeboard. - There are no records of maximum wave heights at existing reservoirs in this region. Accordingly, maximum predicted wave heights were determined by the Stevenson-Molitor formula given in Engineer Bulletin, R. & H. No. 9, 1938 of the Chief of Engineers. The maximum fetch is 2.25 miles. The corresponding maximum wave height is 3.2 feet, for a wind velocity of 60 miles per hour. The reservoir extends east from the dam, while the prevailing maximum winds are from the northwest and west. Therefore maximum wave heights are less likely to occur at the dam than at one where the reservoir extends into the direction of prevailing winds. Wave height plus ride-up, computed at 1.4 times the wave height, is 4.5 feet. Wind set-up, computed from the formula

$$S = \frac{V^2 F}{800 D} \cos A$$

in which S = set-up in feet, V = wind velocity in miles per hour, F = fetch in miles, D = depth in feet, and A = angle between wind and fetch, has a maximum value of .4 foot. Because of the small size of the Birch Hill reservoir, and the narrow zone of possible wind directions which could produce appreciable set-up, this item is not considered critical. The top of the embankment will have a freeboard of 5 feet above the maximum surcharge water surface, or 12 feet above the crest of the spillway.

B. LEVELS AND CAPACITIES OF RESERVOIR.

1. For flood control. - The capacity of the Birch Hill Reservoir for flood control was selected as 49,900 acre-feet below a spillway

crest elevation of 852 feet above mean sea level, as shown on Plate No. 25. It is equivalent to a run-off volume of 5.3 inches from the drainage area of 175 square miles above the dam site, or 6.0 inches on the ultimate net drainage area.

2. For conservation. - No conservation pool will be provided.

C. OUTLET HYDRAULICS.

1. Outlet design. - The following factors have been considered in the design of the outlet, which shall be suitably controlled by gates:

a. The outlet design flood is an hypothetical flood with a volume equal to the volume of run-off of 100-year frequency on a stream of equal drainage area in New England.

b. A retarding basin discharge has been computed such that with all gates open, the pool elevation reaches, but does not exceed, the spillway crest.

c. This discharge has been increased by a flexibility factor of 70 percent in order to provide flexibility of operation within a system of reservoirs in the Connecticut River Basin, to obtain the greatest flood reductions at downstream damage centers.

d. The outlets have been designed to pass the retarding basin discharge increased by the flexibility factor, and further increased to provide for passing 85 percent of the design discharge with one gate closed.

e. Consideration has been given to increasing this design discharge to allow:

(1) Passing of local freshets that do not produce damage, without utilizing more than a minor portion of the flood control

capacity of the reservoir.

(2) Emptying the full reservoir within the period of a few days.

(3) Passing possible minor floods during construction of the dam with upstream levels that will not require an excessive height of cofferdam.

f. The size of the outlet has not been considered as affecting the safety of the dam against overtopping, because in determining the size of the spillway, no outlet discharge is assumed.

2. Satisfaction of design conditions.

a. Outlet design flood. - The outlet design flood adopted has a total volume equivalent to 9.15 inches of depth on the drainage area of 175 square miles. The flood hydrograph was constructed by the unit hydrograph method from a 2-1/2-day rainfall, using the unit graph derived from the September 1938 composite flood. The time-intensity of precipitation assumed was a constantly increasing rate during the first half of the storm and a constantly decreasing rate during the second half. The resulting maximum flood discharge is 17,600 cubic feet per second. This value is 10 percent in excess of the peak discharge of the September 1938 flood, the highest of record. The estimated probable frequency of occurrence of the outlet design flood is once in about 100 years.

b. Determination of outlet capacity. - It was computed by trial and error that an outlet with a discharge capacity of 5400 cubic feet per second, when the water surface of the reservoir is at spillway crest would pass the outlet design flood and that the reservoir would just be filled to spillway crest provided that the gates remained open. The most probable method of operation would be to pass a large portion of the first part of the flood, and store the latter part of the flood in order to get the greatest flood-reducing effect on the Connecticut River. Such an operation would necessitate greater outlet discharge both before and after the throttling period in order to keep the pool at or below spillway crest. Accordingly, a flexibility factor of 1.7 was applied to the value of 5400 cubic feet per second value to obtain the desired maximum outlet capacity of 9200 cubic feet per second. An additional requirement was imposed that 85 percent of the desired capacity or 7820 cubic feet per second be obtained with one gate closed. The computed discharge capacity of the outlet as designed, with the water surface at spillway crest, is 10,500 cubic feet per second with all gates opened, and 7820 cubic feet per second with one gate closed. The natural and modified outlet design flood hydrographs are shown on Plate No. 26. The excess capacity of the outlet as designed is adequate to compensate for any slight decrease in the net flood control capacity due to neglect of the natural valley storage.

c. Adequacy of outlets. - A check was made by determining the greatest pool elevations that would have occurred if the September 1938 flood, the highest of record, had discharged into the empty reservoir. This flood had a peak discharge of 16,000 cubic feet per second with a duration of 8 days, and its total run-off volume was equivalent to 7.5 inches of depth on the drainage area above the dam site. With all gates opened the maximum pool elevation would have been 842.6 feet; with one gate closed 845.1 feet; with two gates closed 848.1 feet. In actual operation the gates would have been throttled considerably to fully utilize the reservoir storage.

d. Gates. - Four sluice gates, each 6 feet by 12 feet, will be provided to take care of desirable variations of discharge during flood periods. The outlets are placed at a low elevation in order to completely drain the pool and effectively utilize the storage capacity. This also affords greater freedom from trouble due to ice and drift, etc. The discharge from the reservoir may be varied from zero to about 10,500 cubic feet per second (reservoir full).

e. Discharge of freshets. - There will be many freshets and small local floods which may occur at any time during the entire year for which storage for flood control is unnecessary. It is desirable that these be passed through the outlets without raising the water surface appreciably higher than it would have been under natural conditions. A maximum peak discharge of 1800 cubic feet per second and a flood run-off of about 2 inches is estimated to be equalled or exceeded an average of once each year. With the gates open such a flood would fill the pool to elevation 826.4 or 8.2 feet above the natural elevation for this discharge at the entrance to the outlet. These heights are predicated upon operation for minimum rises in water surface with all gates fully opened. With a natural river slope upstream from the dam of about 1.65 feet per mile, these rises in elevation will disappear in 5 miles.

f. Time required to empty reservoir. - The time required to empty the reservoir from spillway crest elevation with the outlets fully open and a natural inflow of 2 cubic feet per second per square mile is about seven days. By this operation the channel capacity of the Millers River would be exceeded. By regulating the outlet discharge to channel capacity, the reservoir can be emptied in about 8-1/2 days. If the Connecticut River discharges are near channel capacity, the outlet discharge may be regulated still further and the time of emptying correspondingly increased.

g. Provisions for maximum flood discharge during construction. - During a single construction season it will be necessary to divert the natural flow of the stream through the outlet. A maximum flow of 3500 cubic feet per second, or 19.9 cubic feet per second per square mile, has been selected for construction diversion. It has a probable all-year frequency of recurrence of once in 6.6 years, a May through October construction season frequency of once in 21 years, and a May through December construction season frequency of once in sixteen years. The outlet as designed will discharge 3500 cubic feet per second with the pool at elevation 830 feet. An upstream cofferdam built to elevation 832 should provide ample protection during the construction season. The corresponding elevation for a downstream cofferdam is 823 feet.

3. Operation of reservoir. - For the greater floods the reservoir will be operated so as to secure the greatest possible benefit at main damage centers on the Connecticut River. The outlet will consist of four sluice gates. These gates will normally be open and the reservoir consequently will be empty, with the discharge regulating itself by small variations in depth over the sill of the outlet. When flood retention is required, the inflow may be entirely retained for all floods whose volumes are insufficient to fill the reservoir (5.32 inches). This will

be possible for all but the greater floods. The operation is to be guided by observation of rainfall and stream flow as a flood progresses, so as to determine at the earliest possible time whether a flood can be expected to attain so great a magnitude that it cannot be retained entirely, and how large a flow must be passed through the outlets so as to use all the flood control capacity without overflowing the spillway crest.

D. SPILLWAY HYDRAULICS.

1. Requirements for spillway design. - The spillway shall have sufficient capacity to pass the spillway design flood, which is 35 percent greater than the computed spillway flood, with no possibility of overtopping the dam even under the following adverse conditions:

a. The reservoir filled to spillway crest at the beginning of the spillway design flood.

b. The outlet gates closed.

c. The gates inoperative or the outlet passages blocked during the entire flood period.

d. The maximum possible wave height occurring at the instant of maximum spillway discharge.

2. Spillway design flood. - The spillway design flood was established at 35 percent in excess of the computed spillway flood. The severest possible conditions were assumed in deriving the computed spillway flood and a factor of safety of 1.35 is considered ample. The increase in the computed flood was made by increasing the ordinates of discharge of the hydrograph without changing the duration of the storm. Peak discharge and flood volume were thus increased 35 percent, and the flood period remained unchanged. The peak discharge becomes 51,600 cubic feet

per second and the total volume is 20.66 inches, as shown on Plate No. 27. The peak discharge is 294 cubic feet per second per square mile, or 3900 times the square root of the drainage area in square miles. It should be noted that there is an appreciable factor of safety obtained by the use of a composite hydrograph for the September 1938 flood, from which the unit graph was derived.

3. Spillway surcharge. - Discharge and cost relations. - The spillway design flood was routed through surcharge storage for varying spillway surcharges to determine the relation between maximum surcharge and required spillway discharge capacity. The routing process makes use of the net storage; i. e., it makes allowance for the valley storage which would have been effective under natural conditions. The results of this routing are shown on Plate No. 28. To determine the most economical surcharge the annual costs of dam and spillway were determined separately for varying maximum surcharges and then totaled to produce the graph of maximum surcharge versus total annual cost of reservoir. These graphs are shown on Plate No. 29. From these graphs and other pertinent factors the maximum surcharge of seven feet was selected for design. The spillway discharge required with a surcharge of seven feet is 56,600 cubic feet per second. The surcharge storage, for a surcharge of seven feet, is equivalent to 2.8 inches of run-off on the gross drainage area, and effects a considerable reduction in the inflow hydrograph of the spillway design flood. The freeboard storage, above the surcharge storage, is equivalent to 2.4 inches of run-off on the watershed. The total storage, from spillway crest to top of dam, equals 5.2 inches of run-off, or 25 percent of the spillway design flood volume, or 34 percent of the computed spillway

flood volume. The hydrograph of spillway discharge for the spillway design flood with a maximum surcharge of seven feet is shown on Plate No. 27.

4. Spillway overload capacity. - The computed spillway flood, and floods greater than this by 35 percent and 150 percent, were routed through surcharge storage to produce the curves shown on Plate No. 30. A flood having a volume of 44 inches and a peak discharge of 110,000 cubic feet per second would raise the reservoir level to the top of the dam.

V. LABORATORY AND FIELD INVESTIGATIONS OF SOILS FOR EMBANKMENT AND FOUNDATIONS.

V. LABORATORY AND FIELD INVESTIGATION OF SOILS FOR EMBANKMENT AND FOUNDATIONS.

A. CLASSIFICATION OF MATERIALS. - The Providence District has adopted a convenient system of soil classification having rigidly standardized terms. In this classification soils are divided into 16 classes as shown graphically on Plate No. 31 and described in Table No. 1. The particular features of this system are (1) it recognizes that accumulations of sediments in nature may be either slight mixtures of various grain sizes, relatively uniform in this respect, or intergradations of these types, (2) it affords a means of rapidly evaluating relative permeabilities, (3) it aids in showing more accurately the distribution and geologic relations of the overburden, especially of those portions composed on non-uniformly graded materials, and (4) it contributes to a more reliable classification of soils in the field prior to laboratory confirmation. Even numbers are used to designate materials of uniform texture. Odd numbers designate materials of variable texture. The sizes for gravel, sand, silt and clay are the same as those in the so-called M. I. T. classification with the single exception that size demarcation between fine silt and coarse clay is not rigidly held at 0.002 mm. but is allowed to vary from 0.006 mm. to 0.0006 mm. This flexibility is introduced in recognition of the fact that natural depositions of fine-textured sediments, such as glacial silt (rock flour) behave differently from clays despite similarity of grain-size distribution.

B. GRAIN SIZE ANALYSIS. - Grain size curves of samples were obtained by means of sieve and hydrometer analyses. The materials were carefully classified and their occurrence shown on Plate No. 5. Use was made of every sample recovered in order to show as accurately as possible the actual geological occurrence.

C. WATER CONTENT AND VOID RATIO. - Water contents and void ratios were determined on numerous 2" samples recovered from bore holes in abutments of the dam. In addition these tests were run on samples from test pits representative of borrow areas. An attempt was made in two 6" holes to secure undisturbed samples of the fine Class 6 sand in the dam foundation in order to determine the natural density of this deposit. At present this investigation is not complete as it cannot be stated with definite assurance that samples recovered by present equipment are undisturbed. It is planned to continue this investigation with different sampling methods.

D. PERMEABILITY. - Plate No. 32 entitled "Generalized Section" shows average permeabilities for each class of embankment and foundation material. These values were determined from laboratory tests on both disturbed and undisturbed samples.

E. CONSOLIDATION. - Settlement of the dam is not an important problem at this site. The glacial till in both abutments is very compact with natural void ratios of .2 to .35. Very little settlement will be contributed by the 50 foot layer of fine Class 6 sand underlying the valley floor. Based on consolidation tests of this material, ultimate settlement of the dam has been computed and is noted in Section IX, paragraph G.

F. SHEAR. - Investigations to determine shearing resistance of foundation and embankment materials have been run employing the following methods:

(1) DIRECT SHEAR METHOD - All different types of materials occurring in the foundation or to be used in the embankment investigated by direct shear tests allowing complete consolidation under normal loads with horizontal movement at constant rate of 0.06 inches per minute. All materials were found to be cohesionless with angle of internal friction ranging from 32 to 39°. Typical curves for the pervious and impervious embankment materials are shown in Plates Nos. 33 (BHM-G1d) and 34 (BHM-G2d).

(2) TRIAXIAL COMPRESSION METHOD - In order to determine critical void ratios of the fine Class 6 sand occurring in the foundation triaxial tests were performed on this material in the Concord Soils Laboratory of the Boston District. These investigations are being continued.

G. SEEPAGE STUDIES. - Analysis for quantity of seepage has been made by means of the flow net. A model test was also run on a preliminary embankment section.

H. COMPACTION. - Many compaction tests based on the Proctor Analysis procedure and Terzaghi's relative density determination were performed in the laboratory on materials intended for pervious, random and impervious embankment construction. Plates Nos. 35, 36 (C2d), and 37 (C3d) show typical compaction curves for the various classes of embankment material. Compaction characteristics are also summarized in Table No. 2 entitled "Summary of Materials Available".

I. OTHER TESTS - Numerous other tests of less importance included extraction for solubility and specific gravity.

J. BORROW AREAS. - Six potential borrow areas were explored. Borrow areas finally selected as more suitable are shown on Plate No. 7. Area A is the major source for pervious embankment material, selected pervious filter material, gravel bedding beneath dump rock on slopes and also considered as a potential source for concrete aggregates. Area B is held as an alternate to Area A being less desirable mainly because of its longer haul. Quantities of materials required balanced against materials available and their physical characteristics are summarized in Table No. 2 entitled "Summary of Materials Available". Composite grain size curves for the various types of material are shown on Plates Nos. 38 (BHM-B1d), 39 (B2d), 40 (B3d), 41 (B4d) and 42 (B5d).

PROVIDENCE SOIL CLASSIFICATION
U. S. ENGINEER OFFICE
PROVIDENCE, R. I.
TABLE NO. 1

CLASS	DESCRIPTION OF MATERIAL
1	Graded from Gravel to Coarse Sand. - Contains little medium sand.
2	Coarse to Medium Sand. - Contains little gravel and fine sand.
3	Graded from Gravel to Medium Sand. - Contains little fine sand.
4	Medium to Fine Sand. - Contains little coarse sand and coarse silt.
5	Graded from Gravel to Fine Sand. - Contains little coarse silt.
6	Fine Sand to Coarse Silt. - Contains little medium sand and medium silt.
7	Graded from Gravel to Coarse Silt. - Contains little medium silt.
8	Coarse to Medium Silt. - Contains little fine sand and fine silt.
9	Graded from Gravel to Medium Silt. - Contains little fine silt.
10	Medium to Fine Silt. - Contains little coarse silt and coarse clay. Possesses behavior characteristics of silt.
10 C	Medium Silt to Coarse Clay. - Contains little coarse silt and medium clay. Possesses behavior characteristics of clay.
11	Graded from Gravel or Coarse Sand to Fine Silt. - Contains little coarse clay.
12	Fine Silt to Clay. - Contains little medium silt and fine clay (colloids). Possesses behavior characteristics of silt.
12 C	Clay. - Contains little silt. Possesses behavior characteristics of clay.
13	Graded from Coarse Sand to Clay. - Contains little fine clay (colloids). Possesses behavior characteristics of silt.
13 C	Clay. - Graded from sand to fine clay (colloids). Possesses behavior characteristics of clay.

SOILS LABORATORY
ESTIMATE NO. 5 - TO ACCOMPANY
PRELIMINARY REPORT #6
FEBRUARY 15, 1940

TABLE NO. 1

SUMMARY OF MATERIALS AVAILABLE - BIRCH HILL DAM

MATERIALS REQUIRED: TYPE	MATERIALS REQUIRED: QUANTITY, CU. YDS.	MATERIALS AVAILABLE: SOURCE	QUANTITY, CU. YDS. EXCAV. MEAS.	CLASS AND TYPE	PERMEABILITY COEFFICIENT k = CM./SEC. x 10^{-4}	ANGLE OF INTERNAL FRICTION, ϕ **	COMPACTION CHARACTERISTICS: OPTIMUM WATER CONTENT, %	COMPACTION CHARACTERISTICS: COMPACTED DRY WEIGHT, LBS. PER CU. FT.
Select Impervious Embankment	65,000 Fill Meas. 65,000 Excav. Meas.	Excavation from Outlet Channel (In lower portion of excavation)	49,000	Class 9-7. Glacial till with angular rock fragments. Contains few boulders.	0.1	33° to 39°	7 to 9	130
		May require some processing to remove boulders above 6" in diameter.						
		Excavation Outlet Channel (In upper portion of excavation)	16,000	Class 9-7. Glacial till with many angular rock fragments. Contains many boulders.				
		Will require processing to remove boulders above 6" in diameter.						
Random Embankment	74,000 Fill Meas. 86,000 Excav. Meas.	Excavation from Outlet Channel (At intake and outlet ends on flood plain)	14,000	Class 4, some 2 and 6. Medium silty sand with no boulders.	1 to 10	33° to 34°	12 to 14	110 to 114
		Excavation from Cut-off trench	4,000	Class 4, 6 and 5				
		Spillway Excavation	10,000	Class 7, some 4 and some 2. Glacial till and sand.	0.5 to 15	32° to 34°	10 to 14	108 to 125
		Borrow Area "A" (Random Section)	60,000	Class 7-5 Variable sand and fine gravel.	1 to 10	32° to 34°	12 to 16	110 to 116
Random Impervious Upstream Blanket	28,000 Fill Meas. 25,000 Excav. Meas.	Excavation from Outlet Channel (In upper portion of excavation)	34,000	Class 9-7 Glacial till with many boulders, which are generally concentrated near surface.	3	As placed in spoil blanket without rolling. 32° to 33°	7 to 10	120
					0.2	When compacted by rolling. 33° to 34°	7 to 10	125 to 129
		To be used as a blanket without removing boulders and without rolling. Quantities shown for blanket 200' wide. Width of blanket will be increased to use all available material.						
Pervious Embankment	78,000 Fill Meas. 86,000 Excav. Meas.	Borrow Area "A" (Pervious Section)	180,000	Class 1 and 3 Glacial gravel with 4" to 8" cobbles.	20 to 100	35° to 36°	---	130
Gravel Filter and Bedding	2,800	Borrow Area "A" by selection.						
Concrete Aggregates Coarse Aggregate Fine Aggregate	14,000 7,000	Borrow Area "A" by processing.	Ample	Class 1 and 3 Glacial gravel with 4" to 8" cobbles.				
Roadway Gravel	2,000							
Spoil as fill in upstream river channel	---	Stripping under dam site.	36,000	Topsoil, Cl. 6 sand. Also deposit of organic material.				
		Stripping from Inlet End of Channel Excavation.	9,000	Topsoil.				
Spoil to fill in marsh at end of outlet channel.	---	Stripping from Outlet End of Channel Excavation.	7,000	Topsoil.				
Backfill and spoil at spillways.	---	Spillway Excavation	20,000	Class 7 and 4 and considerable organic material.				
Dumped Rock and	38,000	Excavation from Outlet channel.	54,000 in place with bulking factor of 1.4 provides 75,500	Granite and mica schist. Some badly weathered, to be spoiled.				
Handplaced Riprap	2,800							

** All materials cohesionless - c = 0

VI. DESIGN CRITERIA

VI. DESIGN CRITERIA

The Birch Hill Dam is designed for safety and stability during all conditions of run-off from the drainage area above. A demonstration of the various loading, stability and safety requirement conditions to which the dam will be subjected and its ability to resist those conditions are indicated in Section VII and the following by showing how the dam meets the following design criteria:

(1) The spillway capacity is so great that there is no danger of overtopping even with the outlet closed and the reservoir full when the spillway flood occurs.

(2) The freeboard is so great that there is no danger of overtopping by waves.

(3) The upstream and downstream slopes of the dam are such that with the materials used in construction they will be stable under all conditions.

(4) The line of saturation is well within the downstream toe.

(5) Water which passes through and under the dam will, when it comes to the surface, have a velocity so small that it is incapable of moving any of the material of which the dam or its foundation is composed.

(6) There is no possibility for the free passage of water from the upstream to the downstream face.

(7) No material soluble in water is used in any part of the dam.

(8) The foundation is sufficiently stable to resist undue stresses caused by the embankment load.

VII. GENERAL DESIGN

VII. GENERAL DESIGN

A. STABILITY OF DAM. - In general, design of the embankment is based on experience and comparison with existing rolled-fill dams with consideration given to utilizing available materials from suitable borrow areas and required excavations. The typical maximum section and profile of the dam so designed are shown on Plate No. 43 entitled "Embankment" Details. In this section of the Analysis of Design stability of the embankment above the foundation only has been analyzed. For a more general description of the adopted design and particularly for stability analysis of the foundation reference is made to Section IX, Paragraphs C, D and G.

Shear tests performed on representative remoulded samples of materials intended for use in the embankment resulted in values for the angle of shearing resistance, ϕ, ranging from 32° to 39°. In general tests on impervious and finer-grained materials obtained the higher angle. This is due to the sharp, angular shape of the grains in contrast to the roughly round shape of the particles comprising the more pervious sediments. None of the materials tested possessed cohesion. Typical shear curves are shown on Plates Nos. 33 and 34.

The method used for solution of the stability problem is contained in a paper by Prof. D. W. Taylor of Massachusetts Institute of Technology entitled "Stability of Earth Slopes". This paper was published in the Journal of the Boston Society of Civil Engineers, Volume XXIV, No. 3, July 1937.

The section studied for stability is shown on Plate No. 32. For a dam subject to sudden drawdown the upstream slope is more vulnerable to failure. Hence it will be analyzed in detail for stability. There are four important conditions to which the upstream slope may be subjected:

1. Complete submergence - reservoir full, flood stage to elevation 860.

2. Sudden drawdown - water instantaneously removed.

3. Steady seepage - continuous flow of water through the soil maintained by rain water or means other than the reservoir source.

4. Capillary saturation - no flow of water, no supply, no evaporation.

Other conditions of partial saturation or drying are not so severe and therefore need not be considered. Case 1 is not a critical condition as the shearing strength required for stability will have the lowest value of any of the four conditions stated. Case 4 is the most common for this type of structure and its use as a flood control dam. Studies of Case 3 have shown results in value somewhere between those obtained for Cases 2 and 4. Case 2 is the most important since analysis based on this assumption proved to give the lowest factor of safety. Consequently, only that stability analysis for the embankment when subjected to sudden drawdown (Case 2) will be presented herein.

The assumption of sudden or instantaneous drawdown is extremely severe and is the most unfavorable that can reasonably be considered in calculations. It cannot possibly be met by actual operating conditions. It is assumed further that water within and saturating the upstream section of dam when the upstream slope is completely submerged is retarded appreciably in its attempt to follow drawdown of the reservoir free water level. Such retardation cannot be the case since the pervious outer shells are highly permeable, possessing an overall permeability coefficient of at least 50×10^{-4} cm./sec. and an actual coefficient much in excess of this value near the outer slopes.

The case of complete submergence is transformed into the sudden drawdown case by the instantaneous removal of the force of water against the upstream slope. The weight of soil per cubic foot is now instantaneously increased from a weight for submerged material to a weight for saturated material. The added weight is carried not by the soil skeleton but by imposing an excess of pressure in the water saturating the soil particles. The case of sudden drawdown assumes that this water is percolating outward to the upstream face is retarded long enough to prohibit the building up of frictional resistance between the particles. This phenomenon does not occur in a pervious outer shell. However, the assumption of the worst conditions of seepage are made herein to solve for an absolute minimum value for the factor of safety.

The material intended for use in the pervious outer shells of the dam possesses an average value of $\phi = 35.5°$ when compacted to a void ratio, e = 0.279.

The following nomenclature is used in these slope stability analyses:

ϕ = angle of internal friction, degrees

e = void ratio = $\frac{\text{volume of voids}}{\text{volume of solids}}$

ϕ_d = effective angle of internal friction (ϕ reduced by factor of safety), degrees

i = angle of inclination of side slope with horizontal, degrees

F_r = factor of safety - no cohesion

w = saturated weight of soil, lbs./cu. ft.

H = effective vertical height of slope, ft.

ϕ_w = weighted value of ϕ, degrees

w_o = weight of water = 62.4 lbs./cu. ft.

s = specific gravity of soil = 2.66

w_s = submerged weight of soil, lbs./cu. ft.

All available friction, as interpreted by values of ϕ, was not utilized in the analysis but was reduced by a factor of safety to an effective friction, there remaining an excess of unused friction. Therefore

$$\tan \phi = F_r \tan \phi_d$$

A theoretical determination of ϕ_w, the weighted value of ϕ, at the instant of drawdown, is extremely complicated and lengthy. An arithmetically weighted value, obtained by reducing the effective angle, ϕ_d, by the ratio of the submerged weight to the saturated weight will involve a small discrepancy but is sufficiently accurate to use. Thus

$$\phi_w = \frac{w_s}{w_s+w_o} \phi_d = \frac{s-1}{s+e} \phi_d$$

The section studied for stability, shown on Plate No. 32, assumed side slopes of 1 on 3 or having an angle of inclination, i = 18.4°. Assuming first a factor of safety, $F_r = 1$

$$\phi_d = \frac{\phi}{F_r} \quad \text{or } \phi_d = \phi = 35.5°$$

Now

$$\phi_w = \frac{s-1}{s+e} \phi = \frac{2.66-1}{2.66+0.28} (35.5)$$

$$= 20.1°$$

This means that a side slope of 20.1° can be maintained in this material without sloughing or sliding due to impeded seepage toward the upstream face of the dam when the upstream slope is subjected to sudden drawdown. Or, the maximum slope to which this material can be constructed without danger of instability is 20.1° which is a slope of 1 on 2.73.

Now $$i = \phi_w = \frac{s-1}{s+e}\,\phi_d$$

Or $$\phi_d = \frac{s+e}{s-1}\,i = \frac{2.66+0.28}{2.66-1}\,(18.4)$$

$$= 32.3°$$

This is the effective obliquity of stress or effective angle of internal friction, representative of the amount of friction utilized to hold the slope in stability under sudden drawdown.

Since $$\tan \phi_d = \frac{\tan \phi}{F_r}$$

Therefore $$F_r = \frac{\tan 35.5°}{\tan 32.3°} = \frac{0.713}{0.634} = 1.12$$

The factor of safety of the upstream slope against sloughing or sliding assuming instantaneous drawdown, retarded seepage and inappreciable cohesion is 1.12.

It is not necessary to design for as flat a slope on the downstream face as on the upstream face since seepage forces due to sudden drawdown do not materially affect the downstream portion of the dam. Therefore the criterion of complete capillary saturation (Case 4) with no cohesion in the soil is assumed as a basis for this stability analysis.

Then $$i = \phi_d = \frac{\phi}{F_r}$$

where i for the assumed downstream slope of 1 on 2-1/2 = 21.8°

Therefore $$F_r = \frac{\phi}{i} = \frac{35.5}{21.8} = 1.63$$

The downstream slope if constructed to 1 on 2-1/2 will have a factor of safety of 1.63 when subjected to complete capillary saturation.

B. PERCOLATION THROUGH DAM AND FOUNDATIONS. - The section considered in estimating seepage is shown on Plate No. 32 (BHM-Eld). This section shows average coefficients of permeability and generalized underground conditions on the flood plain.

Based on flow net studies for this section seepage has been estimated at 0.00085 cu. ft. per second per lineal foot, the greater portion of which is through the foundation. With due consideration given to the reduced head acting and relatively impervious foundations at the abutments, total seepage for the entire dam has been estimated at 1.0 cu. ft. per second which is equivalent to 650,000 gallons per day. This value is based on maximum water surface meeting the impervious core at elevation 860, eight feet above spillway crest.

As shown on Plate No. 32 (BHM-Eld), material overlying the fine Class 6 sand in the foundation forms a natural filter downstream from the core. By extending the pervious material beneath the downstream random section full advantage is taken of this natural filter. There is then a progressive increase in permeability and in grain size in the direction of flow upward from the fine sand foundation. With the under seepage concentrated through this natural filter under the random section and weighted down by the load of dam above, any danger of piping through the foundation is eliminated. As a further precaution against piping a layer of gravel bedding filter material is used under the downstream rock toe.

VIII. HYDRAULIC DESIGN.

VIII. HYDRAULIC DESIGN.

A. LOCATION, DESCRIPTION AND DESIGN OF DISCHARGE STRUCTURES.

1. Spillway.

a. Location and description. - A small valley north of the right abutment of the dam provides a natural spillway. Bed rock is close to the surface in several natural saddles, and concrete overflow sections can be constructed to any reasonable length required to ensure a small surcharge. Because of the topography, no other type of spillway was considered. A long spillway is desirable to prevent excessive backwater damage from high surcharge in the town of Baldwinville located on the Otter River in the upper region of the reservoir. Three potential sites for the spillway, located in the narrow valley, were investigated, and the most economical was chosen.

The spillway site is located approximately 1000 feet downstream from the center line of the dam. The spillway weir will consist of two concrete overflow sections separated by a natural knoll, and will be constructed on solid rock. The total length of the spillway crest will be approximately 1100 feet. The main spillway, located on the widest and lowest saddle, will be 720 feet long. The auxiliary spillway will consist of a low concrete section 350 feet long located in the adjacent saddle and a concrete section 30 feet long, closing an abandoned railroad cut through solid rock. The crest elevation will be 852.0 m.s.l. The water will approach the weirs through the valley and no excavation is required in the approach channel. The approach to the main spillway is deep while the approach to the auxiliary spillway is shallow. The discharge, after passing the spillway crest, will be carried in a natural gully to the river below the toe of the dam. Three narrow pilot channels have been provided, principally for drainage. For a plan and sections of the spillway see Plate No. 44. In the design of the spillway, the safety of the dam was considered paramount. The spillway was designed to pass a flood equal to the computed spillway flood increased by 35 percent. The spillway approach is separated from the dam by a natural knoll and the discharge channel is located far enough downstream from the toe of the dam to prevent any damage. The spillway was originally designed to have a total length of 920 feet. It was subsequently found feasible to increase the length to 1100 feet, at a small increase in cost. The following discussion, and all plates, pertain to the original design, having a length of 920 feet. The increase of 180 feet in the length provides an added factor of safety, and is in no way detrimental.

b. Spillway design. - The spillways are required to pass 56,000 c.f.s. with a maximum surcharge of 7 feet measured from crest to reservoir water surface.

(1) Control Section. - The crest of each spillway is formed by a concrete ogee-shaped section, the profile of which follows the curve of the underside of the nappe formed by flow over a sharp crested weir under a head of 7 feet and an equal depth of approach. The discharge was computed from the formula:

$$Q = C\ L\ H^{3/2}$$

in which C is the coefficient of discharge, L is the crest length, and H is the height of the energy gradient above the crest.

Since there is to be practically no excavation in both approach and escape channels, the coefficients of discharge will be relatively low until considerable erosion and scour occur during floods. It was considered that values of 3.4 for the primary spillway and 3.1 for the secondary spillways would be conservative figures to use in computing the discharge.

In computing the loss of head in the long and irregular approach channel the value of "n" in Manning's formula was 0.030. The approach will be cleared up to spillway crest elevation.

(2) Spillway rating curve and increase in capacity by encroaching on dam freeboard. - A rating curve for the spillway is given on Plate No. 45. It may be seen from this curve that the spillway discharge will be 124,000 c.f.s., or 122% greater than the design discharge without having the reservoir surface rise above the top of the dam.

2. Outlet.

a. Location and description. - The outlet structure is located on the right bank and is founded on solid rock. (See Plate No. 46.) The topography and rock surface allow of three types of structure. A reinforced concrete conduit passing under the dam with an intake tower located at the upstream toe of the dam and a formal stilling basin at the downstream toe of the dam was considered undesirable because of the possibility of seepage along the surface of the fill and the conduit. A concrete lined tunnel outlet with either an intake tower at the upstream end or a shaft excavated through rock was rejected because of a higher cost than the outlet structure selected. The selection of the open cut outlet channel was made after careful cost analysis and study of safety considerations. The outlet works as adopted consist of a channel excavated partly in earth and partly in rock, a reinforced concrete gate structure constructed against solid rock, and an operating house located directly above.

(1) Intake channel. - The intake channel will be 1500 feet long, and will have a bottom width of 70 feet which will be re-

duced to 40 feet at the gate structure. The side slopes of the channel will be 1 on 2-1/2 in earth and 4 on 1 in rock. The earth slope nearest the dam will be riprapped. The rock slopes will not be lined with concrete except for a short distance near the gate structure, where low concrete retaining walls are necessary to retain the earth fill.

(2) Gate structure. - The gate structure will be located on the centerline of the dam and will consist of a reinforced concrete structure resting on and keyed into solid rock. The height of the structure from sill elevation 815 m.s.l. to the operating floor at elevation 864.5 m.s.l. will be 49.5 feet. Flow through the gate structure will be controlled by four truck-type service gates each 6 feet wide by 12 feet high. Emergency gate slots will be provided directly upstream from the service gates. Three concrete piers extending up and downstream will separate the flow through the gate structure. The gate openings will be lined with semi-steel conduits. There will be no trash racks.

(3) Operating house. - An operating house will be constructed to house the gate operating machinery and other accessories. Access to the gate wells and gates will be through the operating house. The building will be constructed of brick.

(4) Outlet channel. - The short conduit under the gate structure will discharge directly into the 1,150-foot long outlet channel; no formal stilling basin is considered necessary. The channel will flare from a bottom width of 40 feet to 70 feet a short distance downstream from the gate structure. Except at the structure no lining for the channel will be provided. High velocities are expected in the outlet channel, and field operations may reveal the necessity for concrete

lining. The 1 on 2-1/2 earth slopes will be riprapped where necessary. The water will return to the river well below the toe of the dam.

b. Outlet design. - The outlet works discharge water stored temporarily in the reservoir for flood control, the natural stream flow during construction,and the normal low water flows after completion of the reservoir. As shown in Section IV C, Paragraph 2b, the required outlet capacity for flood control operation is 9,200 c.f.s. with the reservoir surface at spillway crest elevation 852 and all gates open. With one gate closed the required outlet capacity is 85% of the capacity with all gates open or 7,800 c.f.s. Preliminary studies indicated that the diversion requirements would not govern in determining the size of gates.

(1) Computation of gate size. - The selection of the proper gate size was made by equating the head available and the sum of the hydraulic losses for various combinations of size and number of gates until a satisfactory solution was obtained. Consideration of the topography and geology fixed the sill elevation at 815 and preliminary computations showed that the roof of the sluice-exit would not be submerged by tailwater. Hence, the head measured from the reservoir surface at spillway crest to the roof elevation at exit was equated to the hydraulic losses, which included the velocity head at exit. It should be noted that trash racks were not considered necessary for this gate structure.

The hydraulic losses were computed on the basis of the following assumptions which are considered good practice:

(a) Friction. - The value of "n" in Manning's formula, $V = \frac{1.486}{n} R^{2/3} S^{1/2}$ was 0.013 for concrete surface 0.045 for rock cut and 0.025 for earth.

(b) Entrance. - Loss = 0.1 h_v where h_v is the velocity head in the sluice.

(c) Gate slots. - Loss = 0.03 h_v where h_v is the velocity head in the sluice.

(d) Sluice-exit. - Abrupt transition loss = 0.5 $(h_{v_1} - h_{v_2})$ where h_{v_1} and h_{v_2} are the velocity heads before and after the abrupt change in sections. Four 6'x12' gates were used in the final layout. The discharge with one gate closed was required to be 7,800 c.f.s. as explained above or 2,600 c.f.s. per gate. The computation was made as follows:

Reservoir water surface	El. 852
Exit-invert	El. 815
Roof at exit invert	El. 827
Gross head to top of sluice at exit = 25.0' = 852-827	
Losses in terms of h_v(velocity head in sluice)	
Entrance	0.10
Gate slots	0.03
Friction	0.08
	0.21
Exit velocity head	1.00
Gross head =	1.21 h_v = 25.0

$$h_v = \frac{25.0}{1.21} = 20.6$$

$$v = 36.4 \text{ ft/sec.}$$

Q for 1 gate = (6x12)36.4 = 2,620 c.f.s.
Q for 3 gates = 7,860 = 85.4% of 9,200 c.f.s.
Q for 4 gates = 10,500 c.f.s.

(2) Design of bellmouth intake and transitions. - The profile of the roof line of the bellmouth intake was made to conform to a quarter ellipse having major and minor axes 1.0 and 2/3 times the height of gate, with the major axis horizontal. This shape is based upon model studies of similar structures.

Transitions in both approach and exit channels were made to provide satisfactory flow conditions.

(3) General hydraulic behavior. - When the reservoir surface is at spillway crest and the gates are fully open, the velocity of flow at the sluice-exit is 36 ft./sec., and increases to a maximum of 40 ft./sec. at the end of the concrete apron where the channel is 40 feet wide with side slopes of 4 on 1. The velocity is continuously reduced by friction in the rock cut to 22 ft./sec. at Station 17 + 00 where the channel flares gradually from a width of 40 feet to 70 feet in a distance of 80 feet. The velocity increases slightly in the flaring portion and the flow remains in the shooting or lower stage until the hydraulic jump occurs at about Station 17 + 80. The velocity is reduced to 8 ft./sec. and flow remains in the streaming or upper stage in the remainder of the escape channel under all conditions of tailwater. The hydraulic jump occurs well within rock cut for all discharges and all heights of tailwater.

(4) Tailwater and outlet rating curves. - The tailwater rating curve (Plate 24) was established by backwater computations, estimated flood discharges and corresponding highwater marks. The tailwater elevation corresponding to a discharge of 10,500 c.f.s. is 828. The tailwater has no effect on flow condition in the escape channels until it reaches elevation 824 since a control section exists at the end of the escape channel for lower tailwater. The outlet rating curve is shown on Plate 47.

B. FREEBOARD. - Computation shows that the maximum height of waves to be expected is approximately 3.2 feet. To include the effect of ride-up on the sloped surface of embankment, an allowance of 1.4 times the total height of waves from crest to trough or 4.5 feet was made. To this was added a wind set-up of 0.4 foot; the resulting freeboard of 4.9 feet provides an adequate factor of safety. Thus, with the adopted freeboard of

5 feet there is no danger of waves overtopping the dam.

C. MODEL STUDIES. - Model studies of the outlet will be made in the near future to check the hydraulic behavior and suggest possible economies in the design.

D. METHODS USED TO REDUCE ERODING VELOCITIES. - No provision has been made to prevent erosion downstream or upstream from the crests of the spillways since it is considered that any erosion which will take place will not affect the safety of the dam and appurtenant structures. Any erosion of the escape channel which accompanies a flood will increase the capacity of the spillways by reducing the backwater effect and thereby increasing the crest coefficients. The downstream toe of the dam is heavily protected by rock riprap.

The rock cut in the escape channel of the outlet works provides a natural stilling device. Riprap protection of the earth portion of the escape channel is provided a sufficient distance downstream from the hydraulic jump to protect the downstream toe of the dam.

IX. EMBANKMENT AND FOUNDATION DESIGN

IX. EMBANKMENT AND FOUNDATION DESIGN

A. MATERIALS AVAILABLE. - Materials available for embankment construction and their suitability have been discussed in Section V, paragraph J and summarized in Table No. 28 entitled "Summary of Materials Available". Embankment material will be obtained largely from two sources, excavation from the discharge channel and Borrow Area A.

B. ECONOMY OF CONSTRUCTION. - Owing to the low height of the dam and the small quantities involved, the most economical construction of the embankment is by the rolled-fill method. This type of construction also makes it possible to utilize a major portion of the structure excavation, which is the source of impervious and random fill, without stockpiling. Stream diversion and railroad relocation can be deferred until the spring of the second construction season without delaying embankment construction and this will be an advantage in reducing flood hazards during construction.

C. DIMENSIONS AND DESCRIPTIONS. - The profile and sections of the dam are shown on Plate No. 43. The top of the dam is at elevation 864. The minimum height of the dam, 56 feet, occurs for a short section across the river bed, which is approximately at elevation 808. The typical maximum section and the one used for stability computation generally are located on the flat flood plain at elevation 820, giving a height of dam of 44 feet. Various designs and sections were studied. The section as finally adopted consists of a selected impervious core with a 4 foot thick blanket extending from the core to the upstream toe. The impervious core is 10 feet wide on top with 4 on 1 side slopes from elevation 864 to the base of the foundation. This is flanked on both sides by a random or transition section with 1 on 1-1/2 slopes. The classification of random fill on the upstream side will approach the impervious and on the downstream side it will approach the pervious type of fill. Pervious outer shells armored with a 3-foot layer of riprap and including a rockfill at the toes will be constructed to complete the full embankment section. The top width of the embankment will be 25 feet, the outer slopes will be 1 on 2-1/2 from top of dam to spillway elevation 852 and 1 on 3 for the remainder on the upstream side and 1 on 2-1/2 from top of dam to the base on the downstream side. A blanket consisting chiefly of spoil material from structure excavation will extend about 200 feet upstream throughout the valley bottom.

As indicated on the profile, a cut-off trench will be excavated for the entire length of the dam. The trench will be excavated through the pervious overlay of the foundation in the valley floor and both abutments and backfilled with selected impervious materials. This construction will cut off seepage through the pervious overlay and will provide bonding of the impervious core to the relatively impervious fine sand in the foundation and the impervious till occurring in the abutments.

The analysis in Section VII based on extensive test data indicates the section to be stable.

D. HEIGHT AND TOP WIDTH OF DAM. - The dam will have a freeboard of 5 feet above the maximum predicted flood level. It is shown in paragraphs IV A (3) and VIII B that the maximum predicted wave height is 3.2 feet and that the run-up of waves on the 1 on 2-1/2 slope may reach

1.7 feet additional or a total of 4.9 feet. The riprap protected slope and the top width of 25 feet, of which the center 16 feet is paved with gravel and bituminous macadam for a roadway and the remaining 9 feet built as an extension of the riprap from the slopes, will effectively protect against any possible damage from spray or wave action.

E. STABILITY OF SLOPES. - Stability of slopes of impervious and pervious sections of the dam has been demonstrated in Section VII, paragraph A.

F. SATURATION LINE. - The estimated line of saturation is shown on Plate No. 32 (BHM-Eld) for the extreme condition of maximum water surface at elevation 860, eight feet above spillway crest. Extension of the pervious section beneath the downstream random section acts as a horizontal drain, drawing the line of saturation rapidly down and keeping it well within the random section.

Following is the procedure used in obtaining the location of the line of saturation:

(1) Conservative assumption was made that coefficient of permeability in a horizontal direction equals four times that in a vertical direction. A transformed section was then drawn for this condition with horizontal dimensions reduced by a transformation factor of 1/2.

(2) As a most extreme condition it was next assumed that impervious and random sections were of equal permeability. For this case of a homogeneous section discharging into a horizontal drain a theoretical solution is available wherein the line of saturation is a parabola, generally called the Kozeny or basic parabola. Location of this parabola

was accomplished by simple graphical construction.

(3) As a condition at the opposite extreme, it was next assumed that the random section was infinitely pervious. For this condition the exit point for the line of saturation in the core was determined by graphical solution of the L. Casagrande equation for line of saturation - see A. Casagrande, "Seepage Through Dams", Journal of New England Waterworks Association, June 1937.

(4) With extreme limits located by the two preceding steps the position of the line of saturation was then located to allow for actual difference in permeability between core and random sections. The final position was adjusted to equalize approximately the quantity of flow passing through the two sections.

(5) The position of the line of saturation as located in the transformed section was then transferred back to the true scale section, Plate No. 32.

Location of line of saturation is so far within the downstream face of the dam that the possibility of seepage erosion on downstream face is absolutely eliminated.

G. STABILITY OF FOUNDATION. - The section showing generalized foundation conditions on valley floor is presented in Plate No. 32. The principal foundation deposit is a layer of uniformly graded fine sand, Class 6, 60 feet maximum thickness at the valley center, lensing out and disappearing at both valley walls as shown on Plate Nos. 5 and 6. The thickness of this sand layer has been explored by numerous 2-1/2 inch borings, and in two 6" holes (BH-80A and BH-81A) attempts were made to secure undisturbed samples to determine natural density of the layer.

Two types of sampling equipment were employed: a split spoon equipped with spring core catcher like that used in the Denison District for sampling cohesionless soils and a spoon developed by Dr. Piggot for explosion-type sampling. The Piggot spoon was advanced by driving and not by explosion. While the Piggot spoon gave the better results, in neither case is it certain that the samples recovered are reasonably undisturbed. Additional equipment and improved methods of sample recovery are being planned to investigate this sand layer further.

Foundation conditions in both abutments are excellent, a compact glacial till, Classes 9 and 7, overlying rock. This material is very compact, natural void ratios ranging from 0.23 to 0.35 which is equivalent to an average dry weight of 131 lbs. per cubic foot. Also the abutments are very impervious, the coefficient of permeability being in the range $k = 0.1$ to 2×10^{-4} cm./sec.

No settlement is anticipated on the abutments due to the compact nature of the foundation. On the valley floor an ultimate settlement of 9 inches has been estimated from consolidation tests on the fine Class 6 sand. This sand layer is sufficiently pervious so that this settlement will occur during construction at practically the same rate as load is added by embankment construction.

Tests have been performed using triaxial compression apparatus to determine the critical density of the fine Class 6 sand in the foundation. These tests are being continued. As discussed above investigations are still underway to determine the natural void ratio of this material.

X. STRUCTURAL DESIGN

X. STRUCTURAL DESIGN

A. SPECIFICATIONS FOR STRUCTURAL DESIGN

1. General. - Structural design has been carried out in accordance with standard practice. The specifications contained in the following paragraphs govern the design of concrete, reinforced concrete, and structural steel.

Design has been carried out with attention to both stability and durability, with low concrete stresses maintained throughout and reinforcing steel provided as required for stresses due to loading and to temperature changes. Expansion joints are provided in concrete wall and spillway sections at intervals not exceeding 40 feet.

Structures investigated and designed include the gate structure, outlet channel retaining walls, spillway weir, and access road bridge. Design computations will be found in Appendix A.

2. Loads. - a. Dead loads. - The following unit weights for materials have been used:

Item	Unit weight lbs. per cu. ft.
Concrete	150
Brick	130
Water	62.5
Dry earth	100
Saturated earth	125

Uplift assumptions are discussed in the introduction to the Appendix.

b. Live loads. - H-15 loading was used for the design of the access road bridge.

The live loading on the operating floor of the gate structure was that which would result from the heaviest item of equipment resting on the floor, and amounted to 250 pounds per square foot. Floor beams were designed for a loading of 200 pounds per square foot. The gate pull was 47,000 pounds.

Wind loading on exposed vertical faces of the operating house was assumed at 30 pounds per square foot. Snow load on the roof of the operating house was assumed at 40 pounds per square foot.

c. Earth pressures. - Horizontal earth pressures were computed by the use of Rankine's formula, resulting in an equivalent lateral loading of 35 pounds per square foot for dry earth and 80 pounds per square foot for saturated earth.

3. Structural steel. - The design of steel structures has been governed by the "Standard Specifications for Steel Construction" of the American Institute of Steel Construction. Allowable stress in tension or compression is 18,000 pounds per square inch, and in shear is 13,500 pounds per square inch. For wind loading allowable stresses are increased 33-1/3 percent.

4. Concrete and reinforced concrete. - In general, the design of reinforced concrete was governed by the recommendations of the Joint Committee and the American Concrete Institute.

a. Modulus of elasticity. - The modulus of elasticity of concrete with a required 28-day strength of 3400 pounds per square inch was taken as 1/12th that of steel.

b. Coefficient of expansion. - The coefficient of expansion of steel and concrete is assumed at 0.00006 parts per degree Fahrenheit.

c. Allowable working stresses. -

	Lbs per sq. in.
(1) Flexure. - Extreme fiber stress in compression	1000
(2) Shear. - Beams with no web reinforcement and without special anchorage of longitudinal steel	50
Beams with no web reinforcement but with special anchorage of longitudinal steel	75
Beams with properly designed web reinforcement but without special anchorage of longitudinal steel	150
Beams with properly designed web reinforcement and with special anchorage of longitudinal steel	225
(3) Bond. - In beams and slabs (Deformed bars used throughout) (Where special anchorage is provided double these values in bond may be used)	125
(4) Bearing. - Where a concrete member has an area at least twice the area in bearing	625
(5) Axial compression. - In columns with lateral ties	563

(6) Allowable unit stresses in reinforcement

	Ordinary Structures	Structures much exposed to water
Tension	18,000	16,000
Web reinforcement	16,000	16,000

d. Design for surcharge head. - In investigating the gate structure for loading with hydrostatic head to surcharge elevation in the reservoir, allowable unit stresses were increased 33-1/3 percent due to the infrequency and to the short duration of such a condition.

e. Protective concrete covering. -

Type of member	Minimum cover in inches
Roof slabs	1
Floor slabs	2
Beams	2
Heavy members exposed to water and weather	4
Heavy members exposed to abrasive action of flowing water	6
Members, adjacent to rock surface	4

B. DESCRIPTION OF STRUCTURES

1. Outlet Works. - a. General description. - The outlet works will consist of a channel cut into rock at the right end of the dam with reservoir discharges either uncontrolled when functioning automatically as a retarding basin or at flood times, controlled by four 6 by 12 feet truck gates. The gates will be housed in a reinforced concrete gate structure, providing vertical gate wells from the intake passages to the operating floor. The operating house superstructure will consist of a structural steel framework with brick walls and glass brick windows. Gravity section retaining walls will extend from the gate structure along

the channel to retain and protect the earthwork of the dam. The channel will be lined with concrete for a short distance up and downstream from the gate structure.

b. Gate structures. - (1) Description. - The gate tower will be located across the outlet channel at the center line of dam, having rock side walls to approximately Elevation 852.0. There will be four gate passages, each 6 by 12 feet, with floor at Elevation 815.0. Piers will be four feet thick, except for gate recesses, and curtain walls for the gate shafts will be 1 foot 6 inches and 2 feet thick. The operating floor will be at Elevation 864.5 with access by a road over the dam. There will be a small basement at Elevation 853.0, reached by a ladder from the operating floor.

(2) Stability. - The gate structure is designed for stability under conditions resulting from reservoir level at surcharge elevation, which is the maximum condition for overturning. No allowance is made for restraint due to frictional resistance of the rock side walls. Maximum bearing on the rock foundation is 4,000 pounds per square foot. The resultant of forces is maintained within the middle third. Principal members are thickened to provide an additional factor of safety.

(3) Intake passages. - There will be four intake passages, with floor at Elevation 815.0. The floor is designed for bearing loads due to weight of structure and to effect of hydrostatic head against the gate structure. The piers are designed for differential in hydrostatic head existing with flow in some passages with adjacent passages closed. Side walls are designed for 50 percent effective hydrostatic pressure through the rock against the concrete lining. A steel lining is provided downstream from the gates.

(4) Gate wells. - The side walls are designed for 50 percent effective hydrostatic head through the rock. Piers and curtain walls are designed against hydrostatic loading resulting from water in the reservoir to spillway elevation. Members are also investigated for loads with water to surcharge elevation, using 33-1/3 percent increase in allowable unit stresses.

(5) Operating floor. - The floor is designed for heaviest concentration of live load resulting from placing any piece of equipment on it. Top steel is lowered to allow room for electrical conduits above the steel.

(6) Operating house. - The operating house will be a brick superstructure, 22 feet 4 inches by 48 feet 4 inches inside, with 17-inch thick walls. The roof will consist of a 5-inch reinforced concrete slab, supported by steel beams, with a cinder concrete fill, sloped to drain. Crane girders were designed to carry loadings resulting from operation of the emergency gate, which is the maximum loading condition. The roof loads, crane loads, and wind loads will be carried by a structural steel framework of beams and columns. Roof loading consists of a 40-pound per square foot snow load. Wind loading is 30 pounds per square foot of vertical projection of exposed faces.

c. Retaining walls. - Retaining walls of gravity section will extend upstream and downstream from the gate structure, maintaining earth fill adjacent to the channel. They are founded on rock. An 18-inch reinforced concrete lining will be used to protect the channel side walls and floor adjacent to the walls. The retaining walls are designed for stability against earth pressures with earth saturated to the elevation of maximum seepage line through the embankment. Uplift is assumed as 50 percent of effective hydrostatic head. The lining will be anchored to the rock by dowels spaced approximately 8 feet on centers.

2. Spillway. - Two spillway weirs consisting of gravity overflow sections are located in saddles on the right side of the valley. The finished form, dimensions, elevation and gradient of the two spillways are based upon hydraulic considerations. Both spillways have been analyzed for stability. With water to surcharge elevation the resultant falls within the middle third. The maximum bearing pressure is 2700 pounds per square foot.

3. Access road bridge. - The access road bridge consists of a 24-foot span reinforced concrete girder bridge with 16-foot roadway. The footings, abutments, and wingwalls are of reinforced concrete. The bridge span is designed for H-15 loading as the road is a secondary route, serving the dam alone.

4. Equipment. - a. Gates and hoists. - The flow through the conduits will be controlled during flood operation periods by four truck-type, service gates 6 feet wide by 12 feet high in the clear. The gates will operate in vertical slots and be controlled from the operating house located on top of the intake tower. Emergency gate slots will be provided directly upstream from the service gates. One emergency gate of the same type as the service gates will be employed during such periods when any service gate is inoperative. The gates will open and close under all heads up to and including 44 feet, the maximum hydrostatic head on the gate sill. Individual electric-powered stationary screw hoists of 30,000

pound capacity will be provided to raise and close each service gate at a speed of approximately one foot per minute. The hoists will have push-button control and be equipped with mechanical and electrical safety devices. An overhead crane will be provided to raise the emergency gate at a speed of approximately one foot per minute. The bridge and trolley travel is hand operated and so arranged that the trolley can be moved over any gate slot. The hoist is electric motor powered.

The body of the emergency and service gates will be constructed of structural steel with no member less than one-half inch in thickness or stressed greater than 10,000 pounds per square inch in tension under the maximum hydrostatic head of 44 feet at surcharge elevation. Seals will be bronze and all moving parts of hardened steel. Unit stresses used in the design of steel castings will not exceed one-sixth of the ultimate strength of the steel used. A factor of safety of 15 will be used for castings, subject to severe shock or vibration. Semi-steel conduit linings will be embedded in concrete immediately downstream from the gate frames. The linings will extend to a point 4 feet downstream from the face of the service gate frame and will protect the concrete from cavitation and erosion.

b. Standby unit. - A gasoline engine-driven generator will be provided for emergency operation of the gate hoists and lighting system in the event of failure of commercial power. The unit will consist of a 50 horsepower six-cylinder gasoline engine direct connected to a 220-volt, 3-phase, 25 K.W. generator mounted on a common base. The standby unit will be of ample capacity to start and operate either the crane or one gate hoist.

c. Switchboard. - A free-standing, steel-enclosed, low-voltage dead-front type switchboard will distribute power to the gate hoists, crane, and the lighting panelboard.

All circuit breakers will be the air-break type, manually operated. The generator feeder and the incoming line feeder will be controlled by circuit breakers, rated at 600 volts, 60 cycles, A.C. with an interrupting capacity of 10,000 amperes. Each will be provided with three instantaneous and time-delay magnetic overcurrent trips, and magnetic lockout attachments so that one circuit breaker may not be engaged while the other is across the line.

Each power circuit will be controlled by an individual air circuit breaker on the switchboard, having two thermal and instantaneous magnetic trips per circuit. The motors on the gate hoists and crane will be controlled locally by means of magnetic across-the-line starters, providing running overload protection through overload coils. Under-voltage protection is provided by means of the magnetic holding coils of the starters.

A battery charger located on the switchboard will supply direct-current at 12 volts to the starting battery of the gasoline-electric generating set.

d. Lighting. - Five 220-volt flood lights, located on the parapet and controlled by circuit breakers on the switchboard, will provide adequate lighting for the intake and outlet works and the access road across the top of the dam. A 5 Kva lighting transformer on the switchboard will supply 115 volts to a central lighting panelboard mounted on the wall beside the front door. This panelboard will control all operating house lights, basement lights, and front entrance lights.

XI. CONSTRUCTION PROCEDURE

XI. CONSTRUCTION PROCEDURE

A. SEQUENCE OF OPERATIONS. - It is estimated that the construction period will extend over two working seasons, assuming the contract will be let in the early spring of 1940, which will call for completion on or about November 1, 1941, in time for possible fall floods. The problems of river diversion and railroad relocation are such that a two-season construction period will be required. The following schedule shows the major items of work involved, the approximate quantities and planned construction period of each:

(Table on following page)

MAY 1, 1940 - NOVEMBER 15, 1940

FIRST SEASON

ITEM	FROM	TO	No. of Working Days	QUANTITY	Aver. Daily Rate.	REMARKS
Preparation of site	5/1/40	11/15/40	130	--	--	75% of total
Stripping	5/1/40	6/1/40	20	17,000	850	50% of total
Common Excavation	5/1/40	11/15/40	130	100,000	770	70% of total
Borrow Excavation, "A"	5/15/40	11/15/40	120	40,000	330	35% of total
Embankment	5/15/40	11/15/40	120	108,000	900	45% of total
Dumped rock fill	6/1/40	11/15/40	110	16,000	145	41% of total
Rock Excavation	6/1/40	9/1/40	60	52,000	870	100%
Spillway weir, concrete	7/1/40	11/15/40	90	5,000	55	50% of total
Outlet works, concrete	8/15/40	11/15/40	60	2,300	40	100%
Access Bridge	--	11/15/40	--	--	--	100%

MAY 1, 1941 - NOVEMBER 1, 1941

SECOND SEASON

ITEM	FROM	TO	No. of Working Days	QUANTITY	Aver. Daily Rate.	REMARKS
Preparation of site	5/1/41	6/1/41	20	--	--	25% of total
Stripping	5/1/41	6/1/41	20	16,000	850	50% of total
Common Excavation	5/1/41	7/1/41	40	35,000	880	30% of total
Borrow Excavation, "A"	6/1/41	10/1/41	80	100,000	1250	65% of total
Embankment	6/1/41	10/1/41	80	142,000	1800	55% of total
Dumped rock fill	6/15/41	10/1/41	70	23,000	320	59% of total
Spillway weir, concrete	6/15/41	9/15/41	60	5,000	83	50% of total
Diversion of river and temp. cofferdams	--	6/1/41	--	--	--	--
Operating house, superstructure	--	10/1/41	--	--	--	--
Installation of gates and equipment	--	10/15/41	--	--	--	--
Access road	--	10/1/41	--	--	--	--
Job completed	--	11/1/41	--	--	--	--

1st Season. - Clearing of the dam site and work area will be performed by the Government prior to commencement of the contract work. The contract work during the first construction season will involve stripping and grubbing of the dam site and borrow areas, completion of the outlet channel, including the gate structure, training walls, opening of borrow pits, construction of a portion of the spillway weirs and the access road bridge. Suitable structure excavation will be used as fill for the embankment construction to the limits indicated in the area between the river and the railroad line.

2d Season. - Operations in the second construction season will commence with diversion of the Millers River through the outlet channel. It is expected that the relocated railroad will be in operation by the spring of 1941. The upstream and downstream cofferdams will be completed and the dam constructed to final grade. Other work such as construction of the operating house, installation of gates and accessories, construction of the access road and spillway weirs will be completed during this season. During this period the operator's quarters will be constructed and clearing of the reservoir, including the approach channel of the spillway will be performed under separate contracts. Cleaning-up operations will be concluded.

The various operations to be performed are grouped as follows:

1. Clearing.
2. River diversion.
3. Embankment construction
4. Outlet works, structures, gates, etc.
5. Miscellaneous operations.

1. Clearing. - Clearing operations are divided into clearing and removal of structures within the reservoir area and clearing of the dam site incidental to construction. Clearing of the dam site will be performed by the Government, but removal of structures, fences, etc.will be included under the contract. Clearing of the reservoir area is especially suited for relief labor work or for separate contract, is not urgent in connection with the construction of the dam and involves settlement of questions pertaining to rights-of-way and other negotiations with local authorities. Clearing of the dam site involves removal of wood and brush within the foundation area of the dam, outlet and spillway. The borrow areas on the right bank of the river will also be subject to clearing, and this work will be done under the general contract. There are several frame buildings, stone fences and barbed wire fences within the dam site to be removed.

2. River diversion. - A study of the hydrographs shows that of the summer floods, between May 1 and November 1, the highest flood on record on the Millers River reached 16,000 c.f.s. on September 23, 1938. The next highest summer flood during twenty-four years of continuous records occurred on June 25, 1922 with a peak discharge of 3500 c.f.s. A peak discharge of 3,500 c.f.s. is equivalent to a 20-year estimated seasonal frequency. The outlet, with all gates open, will discharge 3,500 c.f.s. at a pool elevation of 830.0 m.s.l., and elevation 832.0 was selected as the top of a cofferdam protecting the work area against a flood of a 20-year frequency.

The tailwater rating curve shows a water elevation of 820.8 m.s.l. for a 3,500 c.f.s. outlet discharge. A downstream cofferdam will therefore be constructed to elevation 823.0 m.s.l. During the construction of the outlet structure, suitable excavation will be placed in designated areas of the embankment. To prevent undue damage in the work area, care must be taken not to constrict the valley from free flow of possible fall or spring floods. The first operation, after the river has been diverted through the outlet, will be to complete the cofferdams. The upstream cofferdam will be constructed as a part of the permanent structure. It will consist of random material, will be 10 feet wide on top and have 1 on 1-1/2 side slopes. The pervious section of the upstream shoulder, the rock toe and riprap will be constructed simultaneously with the semi-impervious section. This construction, as a unit, will serve as a cofferdam and together with the downstream cofferdam will protect the remaining operations, such as stripping, excavation and backfill of the cut-off trench and embankment construction, against a flood of 3,500 c.f.s. of 20-year frequency.

3. Embankment construction. - Due to the presence of the river and the railroad line in the river valley during the first construction season, only limited work can be performed on the embankment during this period. All clearing, some stripping will be done and suitable excavated material from the outlet works will be placed and rolled in designated areas. It is expected that a large number of boulders will be found in the required excavation and it may be necessary to employ a grizzly to obtain suitable material for the embankment. Some stockpiling may be necessary. Rock for rock toes and riprap will be obtained from the required excavations; some of the rock will be stockpiled. Continued construction of the embankment will begin the

second season as soon as the river has been diverted through the outlet, and the railroad line has been abandoned. The cofferdams will be constructed as described under "River diversion". Use of modern construction equipment and standard methods of construction are contemplated throughout. The rolled fill will be placed by trucks or crawler wagons and rolled by sheeps-foot rollers, generally. Settlement gages will be used to determine the settlement of the foundation. For cross-section of the embankment see Plate No. 43.

4. Outlet works, structures, gates, etc. - The outlet works must be completed by the end of the first construction season in order to allow for river diversion at the beginning of the second. Upon completion of the excavation, the concrete gate structure followed by the retaining walls will be constructed. The semi-steel gate linings in the transition section and the gate guides and the guard angles will be placed simultaneously with the concrete. The gates will be placed in closed position to insure a correct fit and left in this position while the concrete is being poured around the lining. It is not considered necessary to construct cofferdams to permit the operation in the dry; it will be sufficient to leave unexcavated barriers at each end of the outlet channel.

5. Miscellaneous operations. - Structures, such as the access road and bridge, spillway weir, operating house and guard fencing, need not be constructed at any definitely determined time. The access road cannot be constructed until the railroad line has been relocated; it is required that this will be done before the beginning of the second construction season. The operating house will be constructed

of brick. The spillway weir may be constructed at the contractor's option, presumably during the first construction season. Placing of guard fences, landscaping and general clean-up will be finished before November 1, 1942.

B. <u>LABORATORY AND FIELD TESTS DURING CONSTRUCTION</u>. - Many tests have been performed on materials extracted from the foundation and prospective borrow pits both in disturbed and undisturbed form in the U. S. Engineer Soils Laboratory in Providence, Rhode Island. Although a comprehensive picture of conditions in the field has been developed from the results of these tests, it is contemplated that a field laboratory will be established at the site and that tests will be performed during construction to aid in classification of material extracted from borrow pits and structure excavation. The tests will be done to aid in directing and supervising the relative placement of the various types of materials of which the embankment will consist; also for determination of the depth of the cut-off trench. The laboratory, as contemplated, will be supplied with equipment necessary for the following tests: (1) classification of materials; (2) grain size analysis; (3) water content determination of materials in borrow areas and embankment; (4) determination of compacted weight of embankment material in place; and (5) laboratory determination of compaction characteristics of borrow materials. The classification of materials as adopted by the Providence District during preliminary investigations and detailed design of the dam will govern. The embankment will consist of four distinct different types of fill:

1. Pervious fill.

2. Impervious and random fill.

3. Rock toes and dumped riprap.

4. Gravel filters and bedding.

1. Pervious fill. - For location in the embankment see Plate No. 42. Most of this material will be obtained from borrow pit "A". Structure excavations, if found suitable by required tests, may be used in the embankment. It will consist of classes 2, 3, 4 and 5, and shall be placed in such a manner that the finer materials will be nearer the random sections, and the coarser nearer the riprap or outer faces of the embankment.

2. Impervious and random fill. - For location in the embankment see Plate No. 43. Most of this material will be obtained from structure excavation. The impervious material will consist of classes 7 and 9, and the random of classes 2, 4, 6, 7, and 9. The two types of fill will be obtained from structure excavations, supplemented by borrow excavations if necessary, and the material will be placed in the embankment under the direction of Government inspectors. In general, visual inspection of this material will be sufficient, but tests will be performed at frequent intervals to aid in judgment of classification in order to secure the maximum density of the materials in the embankment. Corrections, adjustment and modification of methods may be made from time to time on the basis of those tests.

3. Rock toes and dumped riprap. - For location in the embankment see Plate No. 43. In general, no laboratory tests will be required for the rock used in rock toes and dumped riprap. Although it will be permitted to use boulders and large cobbles from the borrow pits as rock

fill, the major part will be obtained from structure excavations. With the present knowledge of the quality of rock at the site there will be sufficient quantities available, although some selection may be necessary. The rock will be dumped in place with the larger rocks at the outer faces and the smaller rocks and spalls adjacent to the embankment.

4. Gravel filters and bedding. - For location in the embankment see Plate No. 43. The gravel bedding for riprap and rock toes will be obtained from borrow pit "A" and will not be subject to screening. No rigid laboratory tests will be required for this class of material and, in general, visual inspection will be sufficient.

C. PLACEMENT OF TYPES OF MATERIALS.

1. Method of placing and rolling. - After stripping has been completed the foundation will be prepared by plowing followed by rolling and will be ready for application of the rolled fill. The cut-off trench will be filled with impervious material placed in the dry. As soon as the cut-off trench has been filled, the impervious and random sections of the embankment will be brought up to a crown running parallel with the center line of the dam and with the slopes approximately on a 2 percent grade toward the edges of the pervious section. This slope will be maintained until the completion of the embankment. The pervious shoulders will be brought up with the central core and a slight slope from the random section toward the riprap will be maintained until completion.

a. Wetting. - The material will be excavated above the water table. If the water table is high it will be required to be lowered before excavation may take place. Before spreading and rolling, the preceding layer will, in case it has dried out to cause cracks in the surface, be dampened to insure adequate bond. Should the material be too high in water content when dumped, it will be bulldozed or otherwise spread in layers of 6-inch thickness and left for a sufficient time to allow the surplus water to dry before rolling.

b. Spreading. - When two or more different materials are being transported in the embankment they will be placed systematically so that in any area of the section there will be the required proportions of the materials. After dumping, the materials will be bulldozed or otherwise spread in layers of the required thickness, and if the water content is considered correct, it will be rolled immediately. If the preceding layer of materials is too smooth to insure proper bond, it will be roughened or loosened by harrowing.

c. Rolling. - When the moisture content and condition of the spread layer are satisfactory, the rolling will take place. It is required that modern equipment will be used for rolling, such as sheepsfoot tamper type, twin rollers pulled by a crawler type tractor for the impervious and random sections and a plain cylindrical roller for the pervious section.

D. FIELD CONTROL DURING CONSTRUCTION.

1. Field control required. - In order to be assured of a satisfactory result from field operations it is necessary to maintain a complete control at all times. The equipment used must be in good operating condition and no old or inefficient equipment will be allowed. The materials used will be inspected frequently, tested and approved before they are allowed to be placed in the embankment.

a. Compaction. - Rolling of the impervious and random sections will be done by rollers of the sheeps-foot or other satisfactory tamper type roller of sufficient weight. Rolling of the pervious section may be done with a plain cylindrical roller of sufficient weight, a steel disc roller or with the same type of roller as used for impervious fill, depending on conditions.

b. Degree of compaction. - Each square foot of layer of the impervious and random sections will be compacted by not less than six passes of the rollers and ordinarily not more than nine passes, in such a manner that each pass shall adjoin the preceding and adjacent pass. The pervious section will be compacted by six passes of the roller in the same manner as described for impervious fill.

c. Method of testing. - Samples of all embankment materials for testing will be taken both before and after placing and compaction at frequent intervals and tests will be made for classification of materials, grain size analysis, water contents, compacted weights and compaction characteristics. These tests will be made in the field laboratory.

d. Amount of foundation settlement, method of determination, gages, etc. - It is anticipated that some settlement or shrinkage of the foundation will take place as the construction progresses. The amount of settlement is indeterminate beforehand and will be determined by the reading of settlement gages. A total of nine settlement gages will be placed in 5-foot sections. It is believed that the foundation settlement will occur during the construction at the same rate as the load is added by embankment construction, and no settlement is anticipated after completion.

E. <u>**CONCRETE CONSTRUCTION.**</u> - The concrete will be composed of cement, fine aggregate, coarse aggregate and water so proportioned and mixed as to produce a plastic, workable mixture. Two different mixes of concrete will be used throughout, except under special conditions. On the drawings and in the specifications, these types are designated as Class "A" and Class "B". The average compressive strength for Class "A" concrete shall be 3,400 pounds per square inch, and for Class "B" concrete, 3,000 pounds per square inch, in accordance with a standard 28-day test. Concrete aggregates, available at commercial pits within a 15-mile limit, have been tested by the Central Concrete Testing Laboratory at West Point, and found to be of suitable quality. Aggregates from other nearby sources are under investigation. Suitable concrete aggregates are available in Borrow Area "A" and may be used at the contractor's option.

1. <u>Laboratory control</u> - A small concrete testing laboratory will be set up at the site. The tests performed here will supplement those made at the Central Laboratory. The laboratory will be used principally to control the quality of concrete during construction. Facilities will be available for testing the grading of aggregates, designing concrete mixtures, mixing of trial concrete batches for the purpose of developing actual relations between compressive strength and water cement ratio, controlling workability of concrete by slump tests, and casting of concrete cylinders for compressive strength tests.

a. <u>Cement.</u> - Cement will be tested by a recognized testing laboratory and results of these tests shall be known before the cement is used. True Portland cement of a well-known and acceptable brand will be used throughout.

b. Fine aggregate. - Natural sand will be used as fine aggregate, which will be subject to careful, thorough analysis, and tests made on mortar specimens for compressive strength.

c. Coarse aggregate. - Washed gravel or crushed stone of required sizes will be used as coarse aggregate. It must consist of hard, tough and durable particles free from adherent coating and must be free from vegetable matter. Only a small amount of soft, friable, thin or elongated particles will be allowed. The aggregate will be subject to freezing and thawing tests and to careful, thorough analysis, including magnesium sulphate test for soundness.

d. Water. - The amount of water used per bag of cement for each batch of concrete will be predetermined; in general, it will be the minimum amount necessary to produce a plastic mixture of the strength specified. Slump tests will be required in accordance with specifications.

2. Field control. - a. Storage. - The concrete components will be stored separately before mixing. The cement will be stored in a thoroughly dry, weather-tight and properly ventilated building. The fine and coarse aggregates will be stored in such a manner that inclusion of foreign material will be avoided.

b. Mixing. - The exact proportion of all materials in the concrete will be predetermined. The mixing will be done in approved mechanical mixers of a rotating drum type, and there must be adequate facilities for accurate measurement and control of each of the materials used in the concrete. Mixing will be done in batches of sizes as directed and samples will be taken for slump tests and compressive strength tests. Inspectors will at all times supervise and inspect the mixing procedure.

c. Placing. - Concrete will be placed before initial set has occurred. All concrete will be placed upon clean, damp surfaces. Rock surfaces will be free from loose material or other matter interfering with a satisfactory bond. Mass concrete will be placed monolithic. Mechanical vibration will be applied, and forking or hand-spading will be applied adjacent to forms on exposed surfaces in order to insure smooth, even surfaces. Locations of vertical and horizontal construction joints, as well as contraction and expansion joints, are indicated on the drawings. The locations of construction joints are tentative only and may be changed to suit conditions in the field. Before placing the concrete, all reinforcing steel will be inspected and pouring of the concrete will be supervised and directed by Government inspectors. Adequate precautions will be taken if concrete is to be placed in cold or hot weather.

F. **STRUCTURAL STEEL CONSTRUCTION.** - Structural steel construction, other than reinforcement and anchor bars, consists of (1) structural steel for the operating house; and (2) miscellaneous frames, angles, tees and flat steel bars.

1. Structural steel for the operating house. - The operating house consists of a framework of rolled steel columns and roof girders and beams with crane girders carried on the columns. All work will be riveted except base angles and bars which will be shop welded. Erection will be carried out in due order, following construction of the

gate tower concrete substructure. Columns will be encased in concrete to the height of the concrete parapet wall and roof beams will be encased in concrete.

2. Miscellaneous frames, angles, tees and steel bars. - Miscellaneous structural steel, such as metal door frames, guard angles, tees, and flat steel bars embedded in concrete, gratings and grilles, etc., will be erected and placed as indicated on the drawings and at such time as required.

G. **MISCELLANEOUS CONSTRUCTION.** - The operating house will be built of brick with steel lintels, glass brick windows and precast artificial stone door trim.

Other miscellaneous construction items include the construction of the access road, riprapping of earth slopes other than dumped riprap in the outlet channel as indicated on the drawings, erection of guard fencing, landscaping and clean-up job.

XII. SUMMARY OF COST

XII. SUMMARY OF COST

The total construction cost of the Birch Hill Dam, except clearing of reservoir and dam site and construction of the operator's quarters, has been estimated to be $867,000 including 10 percent for contingencies and 15 percent for Engineering and Overhead. This amount has been distributed as follows:

(1)	Embankment	$372,000
(2)	Concrete features	240,000
(3)	Gates and accessories and structural steel	95,000
(4)	Miscellaneous	160,000

(1) The embankment included under Item (1) consists of all earth fill for the dam with cut-off trench, dumped riprap, gravel bedding, rock toes and drains, gravel for road on top of the dam, settlement gages, and as much of the structure excavation as is used in the embankment.

(2) The concrete features included consist of gate structure, including piers and operating house floor, retaining walls with outlet channel lining, spillway weirs and access road bridge.

(3) The steel construction included consists of gates and machinery, and miscellaneous structural steel, except for steel furnished with operating house.

(4) Miscellaneous items included are: operating house (exclusive of the concrete floor), access road, guard railing and fencing, river diversion, clearing of borrow areas and grubbing of the dam site, iron and metal work, pipes and all other items not included under Items (1), (2), and (3).

XIII. INDEX OF PLATES

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

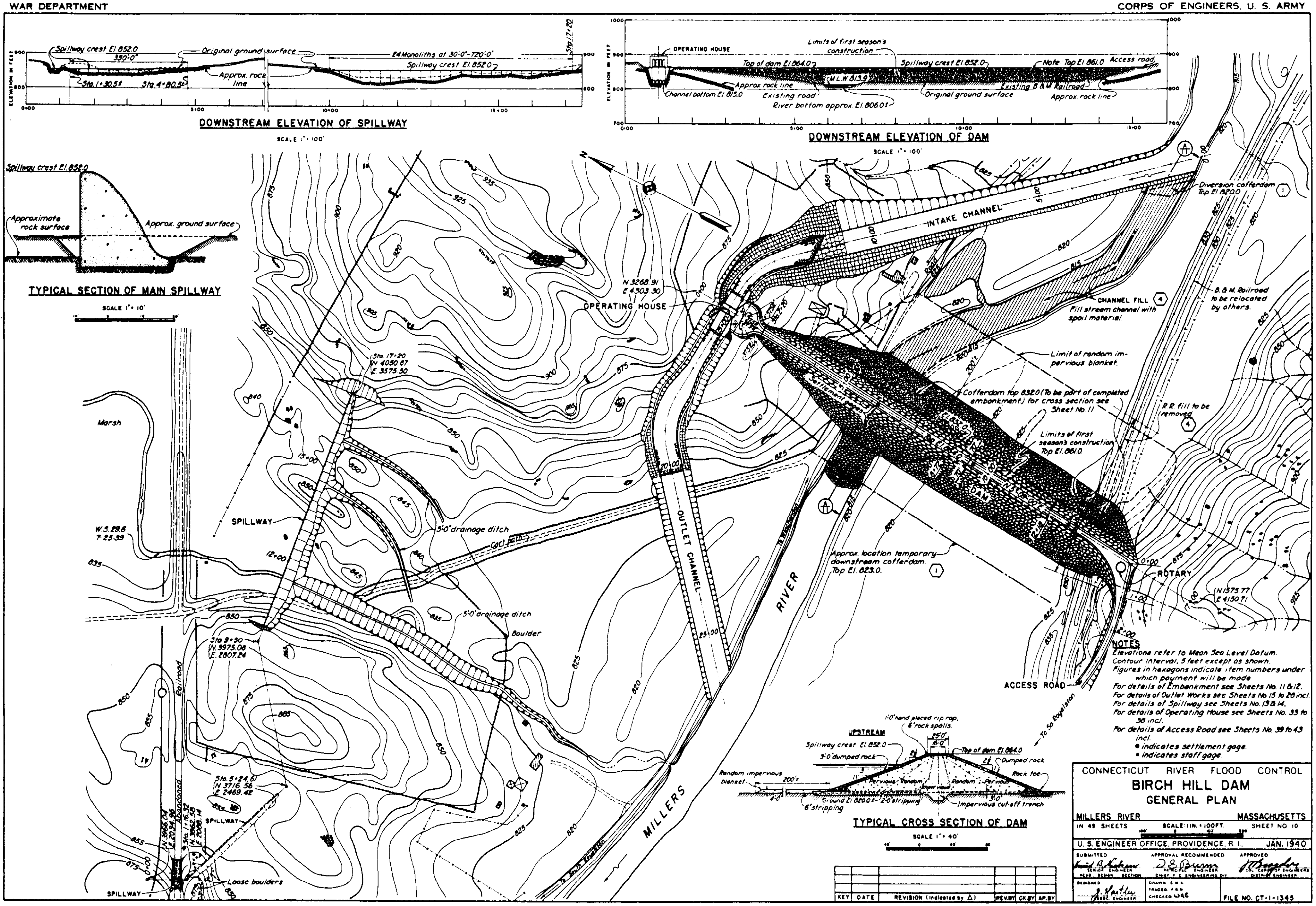
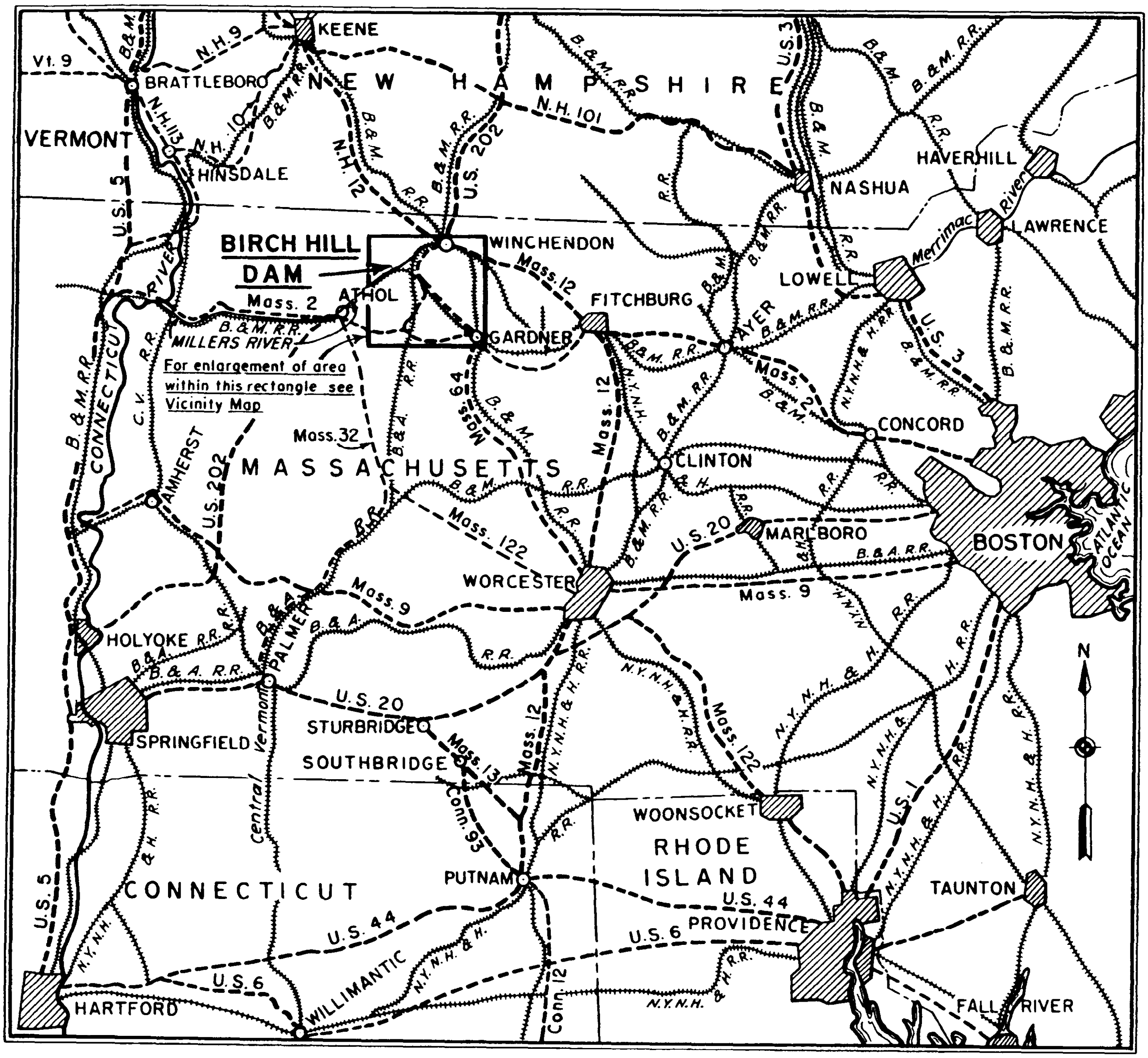


PLATE NO. I

G.49

(10)



LOCATION MAP

SCALE

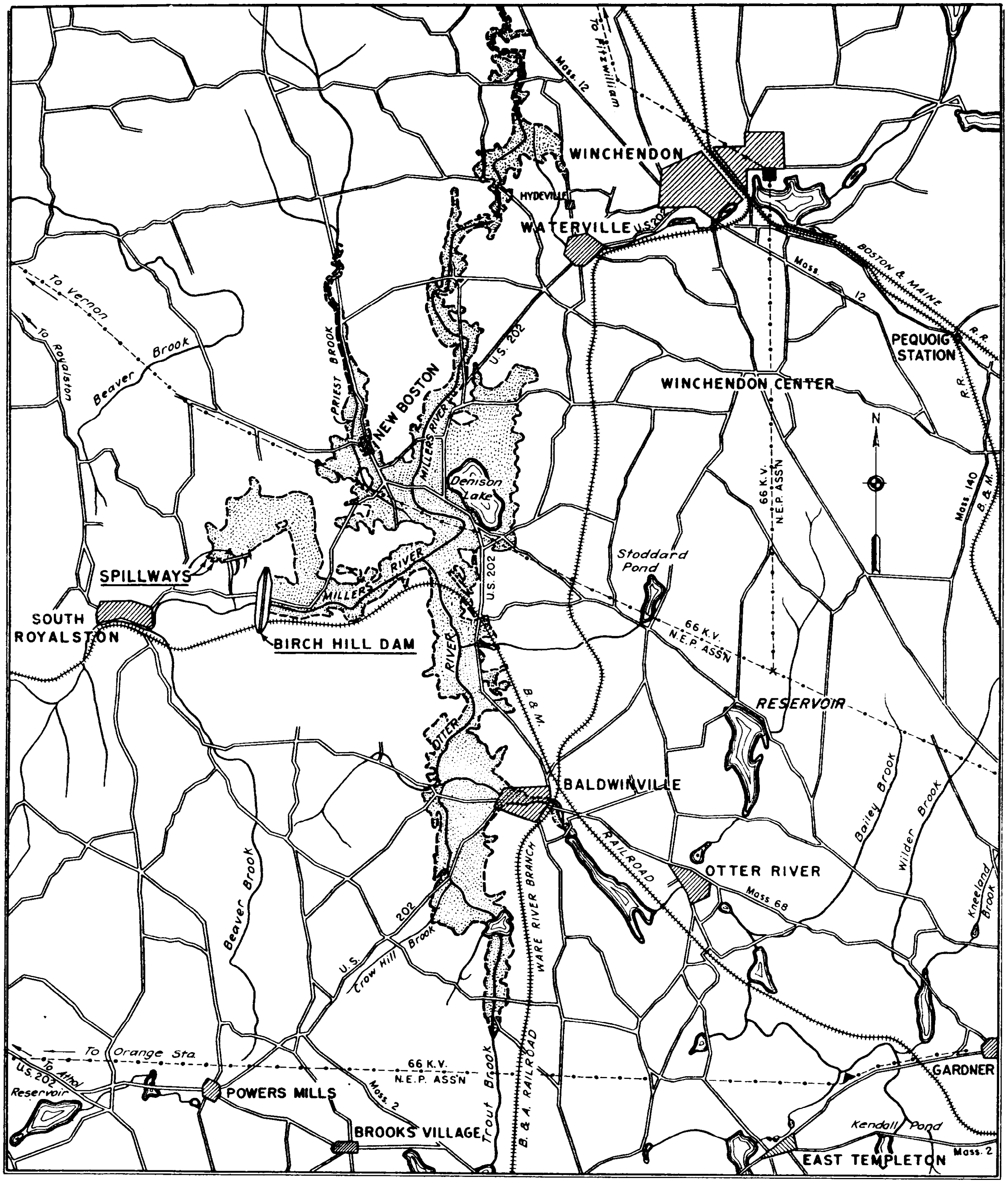
10 0 10 20 MILES

LEGEND

– – – = HIGHWAYS

+++++++ = RAILROADS

PLATE NO. 2



VICINITY MAP

SCALE MILES

LEGEND

- = HIGHWAYS
- = RAILROADS
- = ELECTRIC POWER LINE
- = MAXIMUM RESERVOIR FLOW LINE
- ▲ = ELECTRIC SUBSTATION
- ■ = HYDROELECTRIC GENERATING STATION

PLATE NO. 3

WAR DEPARTMENT

CORPS OF ENGINEERS. U. S. ARMY

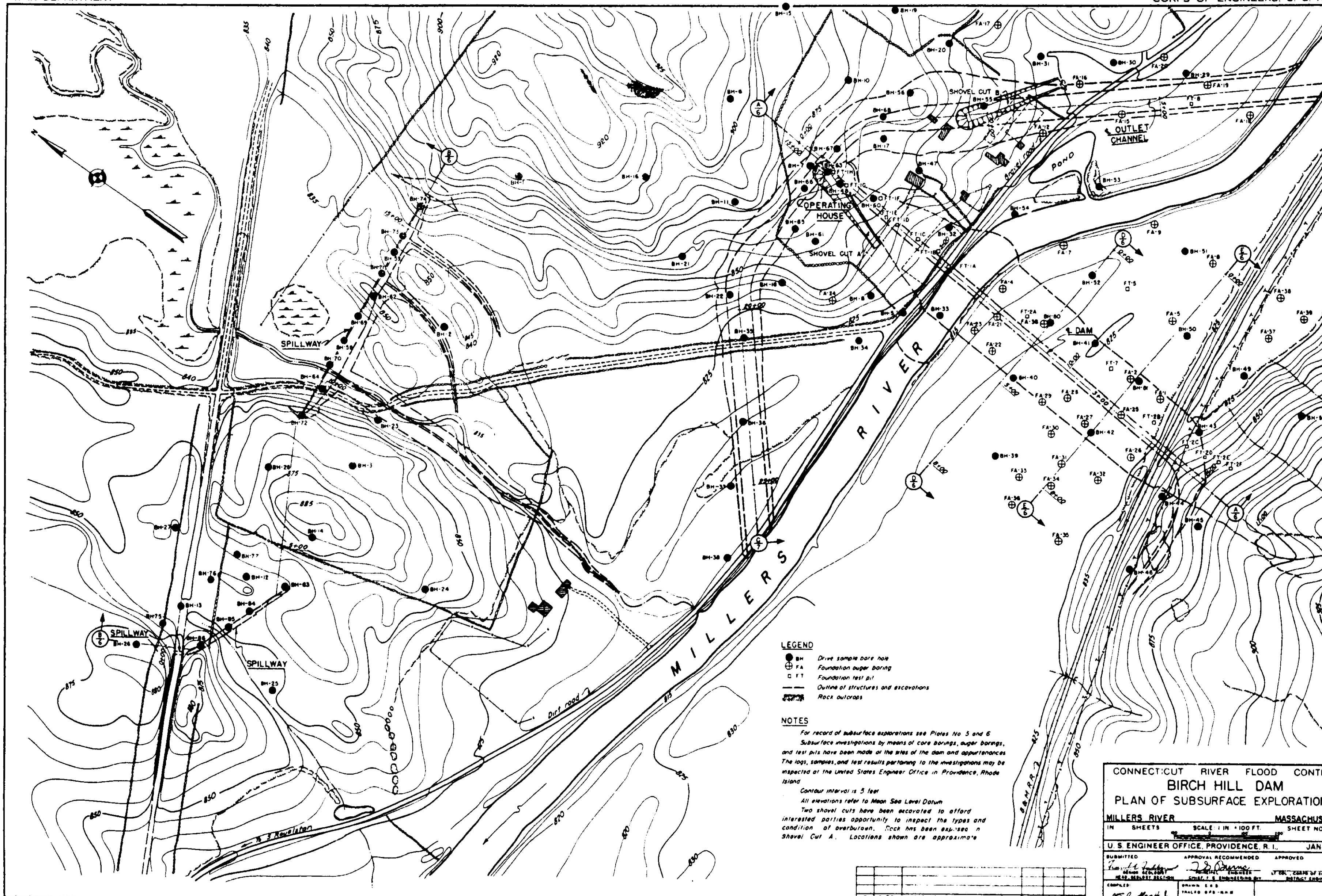
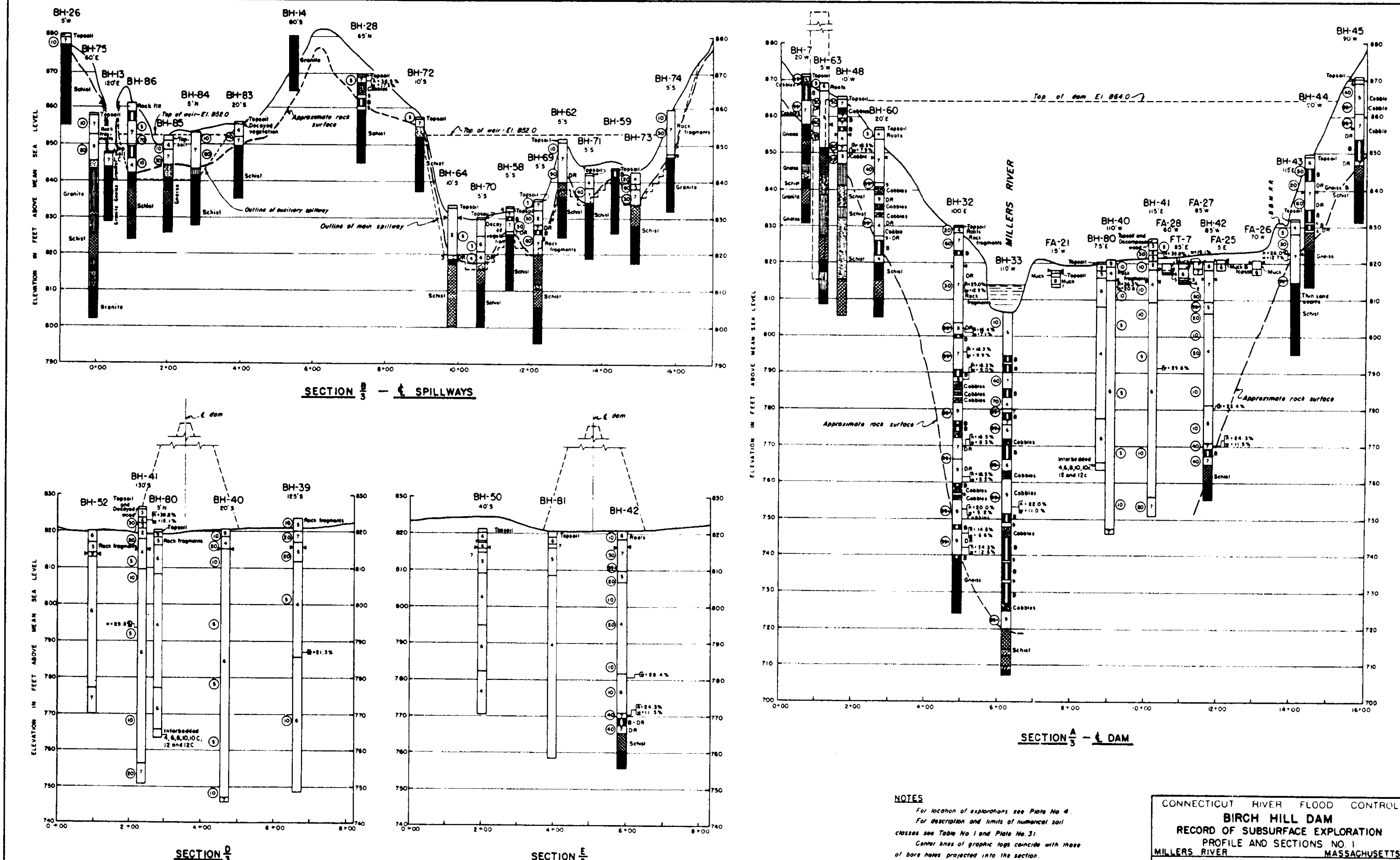


PLATE NO. 4

A. of D. G.49



NOTES

For location of explorations see Plate No. 4.

For description and limits of numerical soil classes see Table No. 1 and Plate No. 31.

Center lines of graphic logs coincide with those of bore holes projected into the section.

N, S, E, or W indicates north, south, east or west of section line.

For general notes and legend see Plate No. 6.

REV	DATE	REVISION (indicated by △)	REV BY	CK BY	AP BY

CONNECTICUT RIVER FLOOD CONTROL

BIRCH HILL DAM

RECORD OF SUBSURFACE EXPLORATION

PROFILE AND SECTIONS NO. 1

MILLERS RIVER — MASSACHUSETTS

IN SHEETS — SCALE AS SHOWN — SHEET NO.

U. S. ENGINEER OFFICE, PROVIDENCE, R. I. — JAN. 1940

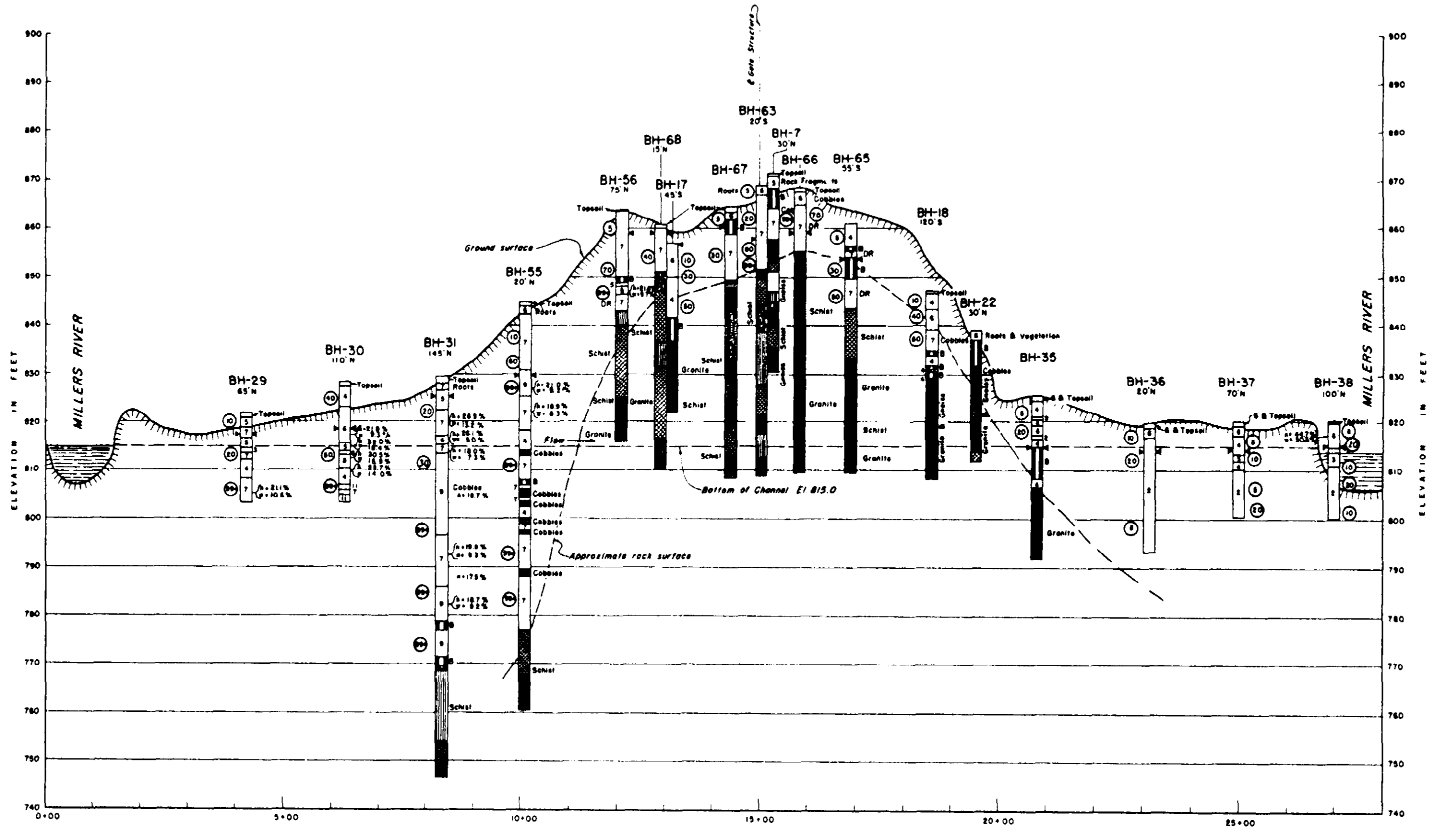
SUBMITTED — APPROVAL RECOMMENDED — APPROVED

SENIOR GEOLOGIST, HEAD GEOLOGY SECTION — PRINCIPAL ENGINEER, CHIEF F. C. ENGINEERING DIV. — LT. COL. CORPS OF ENGINEERS, DISTRICT ENGINEER

COMPILED — ASST. ENGINEER

DRAWN J.M.L. TRACED L.V.P. CHECKED

FILE NO. CT-2-1222



PROFILE ALONG CENTER LINE OF OUTLET CHANNEL — ℄

LEGEND

DR - *Sample contains decayed and disintegrated rock*

- *Water table at time of exploration*

B - *Boulder*

- *Badly weathered or decayed rock*
- *Weathered and fractured bedrock, core recovery between 15% and 50%*
- *Slightly weathered and fractured bedrock, core recovery between 50% and 75%*
- *Relatively fresh or unweathered bedrock, core recovery greater than 75%*

n - *Porosity at depth indicated* = $\frac{\text{Volume of voids}}{\text{Total volume}}$

w - *Water content at depth indicated* = $\frac{\text{Wt. of water}}{\text{Wt. of solids}}$

NOTES

For location of bore holes see Plate No. 4.

Stationing is that of center line of Outlet Channel.

Elevations are referred to Mean Sea Level Datum.

For description and limits of numerical soil classes, see Table No. 1 and Plate No. 31.

Center lines of graphic logs coincide with those of bore holes projected into the section.

N, S, E, or W indicates north, south, east, or west of section line.

Bedrock is composed of mica schist, and granite. These two types frequently occur in large separate units, less frequently as mixtures of both types. The term gneiss is used in the above records to indicate such occurrences.

Numbers in circles indicate number of blows required to drive 2" O.D. sample spoon one foot with 300 pound hammer dropped 18".

KEY	DATE	REVISION (Indicated by Δ)	REV.BY	CK.BY	AP.BY

CONNECTICUT RIVER FLOOD CONTROL

BIRCH HILL DAM

RECORD OF SUBSURFACE EXPLORATION

PROFILE AND SECTIONS NO. 2

MILLERS RIVER MASSACHUSETTS

IN SHEETS SCALE AS SHOWN SHEET NO.

U. S. ENGINEER OFFICE, PROVIDENCE, R. I., JAN. 1940

SUBMITTED: APPROVAL RECOMMENDED: APPROVED:

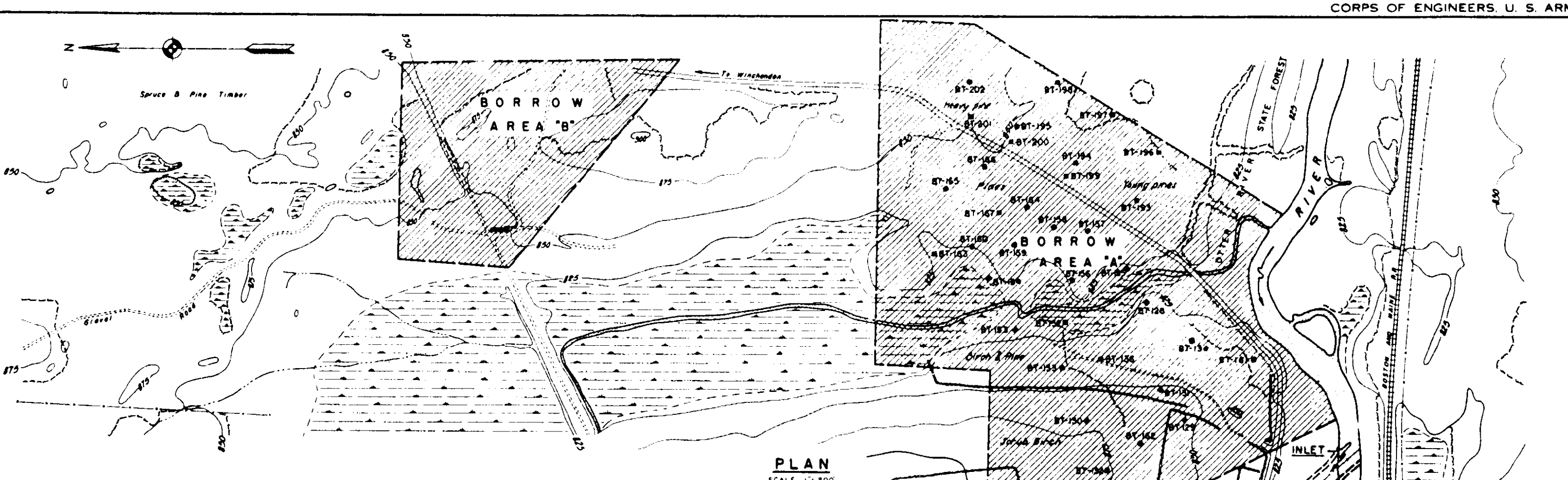
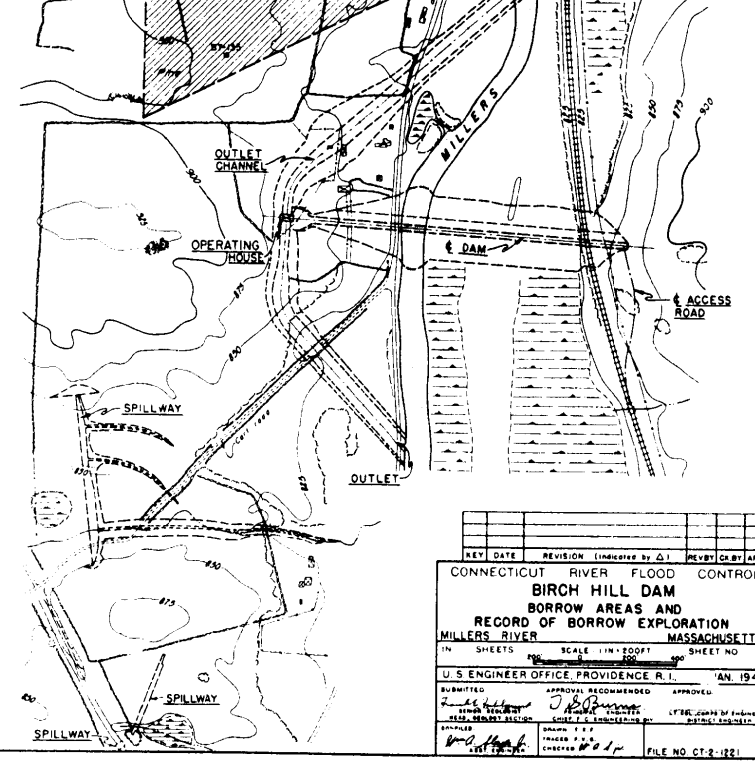
SENIOR GEOLOGIST, HEAD GEOLOGY SECTION — PRINCIPAL ENGINEER, CHIEF, E. & ENGINEERING DIV. — LT. COL., CORPS OF ENGINEERS, DISTRICT ENGINEER

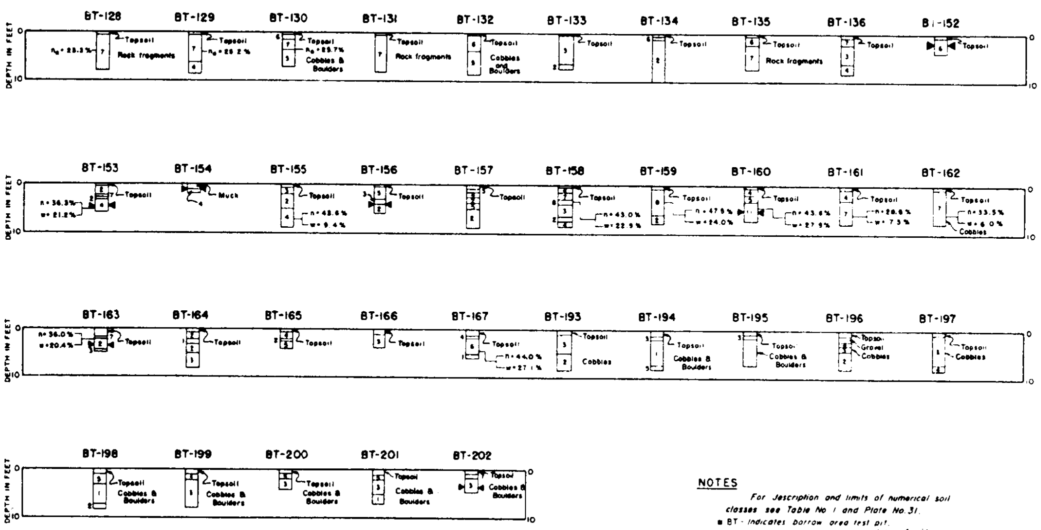
COMPILED: ASST. ENGINEER

DRAWN: J. R. L.
TRACED: L. J. D.
CHECKED:

FILE NO. CT-2-1219

A. of D. G. 49



RECORD OF EXPLORATIONS IN BORROW AREA "A"

NOTES

For Jescription and limits of numerical soil classes see Table No 1 and Plate No. 31.

■ BT - indicates borrow area test pit.

n_a - Average porosity of entire pit = $\frac{\text{volume of voids}}{\text{total volume}}$

n - Porosity at depth indicated = $\frac{\text{volume of voids}}{\text{total volume}}$

w - Water content at depth indicated = $\frac{\text{weight of water}}{\text{weight of solids}}$

BORROW AREA "A"	OUTLET CHANNEL EXCAVATION	SPILLWAY EXCAVATION	BORROW AREA "B"
Principal source for pervious material and supplementary source for random embankment materials. Materials in area on southeasterly side of road composed of coarse sand, gravel and small boulders to a depth of about 8 feet. Deposits are available for pervious embankment construction, selected pervious filter construction and gravel bedding beneath dumped rock. They are also a potential source for concrete aggregates. Overburden in area on northwesterly side of road composed of irregular deposits of sand and gravel, including some cobbles. Selected portions are available for pervious embankment construction, the remaining materials being available c..y for random embankment construction.	Overburden to be excavated composed of sand and gravel including many boulders on and near the surface and compact mixtures of sand, silt and gravel with some boulders and disintegrated rock at greater depth. In places this overburden merges into decayed bedrock. Materials excavated below the stripping line are available for random and impervious embankment construction. Those near the surface may be used in the random section and selected portions obtained at greater depth in the impervious core section	Materials composed of mixed sand, silt, gravel, cobbles and boulders. Available for random embankment construction only	Potential source for concrete aggregate. Will be made available if desired

KEY	DATE	REVISION (indicated by △)	REV.BY	CK.BY	AP.BY

CONNECTICUT RIVER FLOOD CONTROL

BIRCH HILL DAM

BORROW AREAS AND RECORD OF BORROW EXPLORATION

MILLERS RIVER MASSACHUSETTS

IN SHEETS SCALE 1 IN = 200 FT SHEET NO

200' 0 200 400'

U. S. ENGINEER OFFICE, PROVIDENCE, R. I. 'AN. 1940

SUBMITTED: SENIOR GEOLOGIST, HEAD, GEOLOGY SECTION

APPROVAL RECOMMENDED: T. S. Burns, PRINCIPAL ENGINEER, CHIEF, F. C. ENGINEERING DIV.

APPROVED: LT. COL., CORPS OF ENGINEERS, DISTRICT ENGINEER

COMPILED: ASST. ENGINEER

DRAWN F. E. F.
TRACED F. V. S.
CHECKED W. A. S. Jr.

FILE NO. CT-2-1221

PLATE NO. 7

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

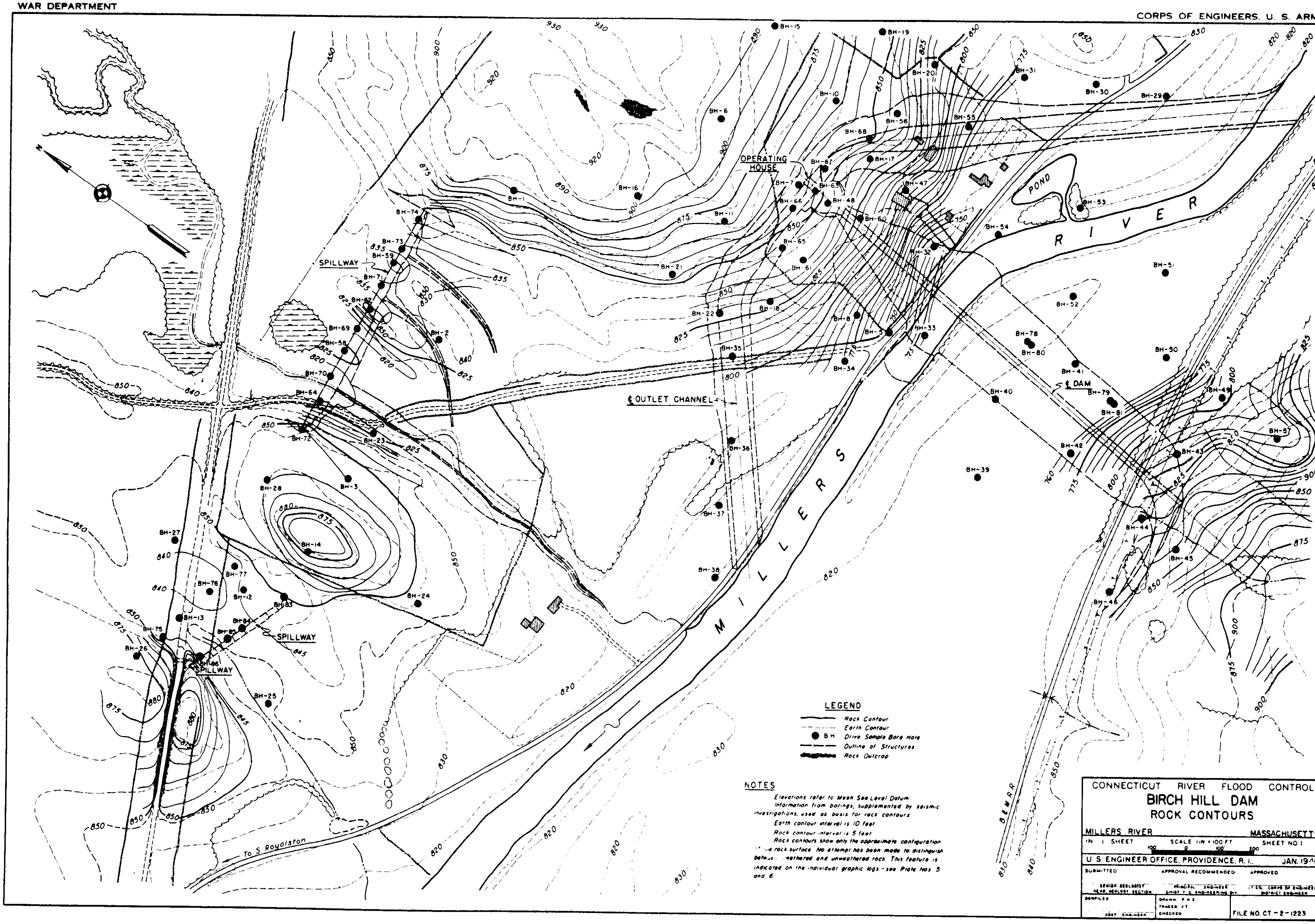
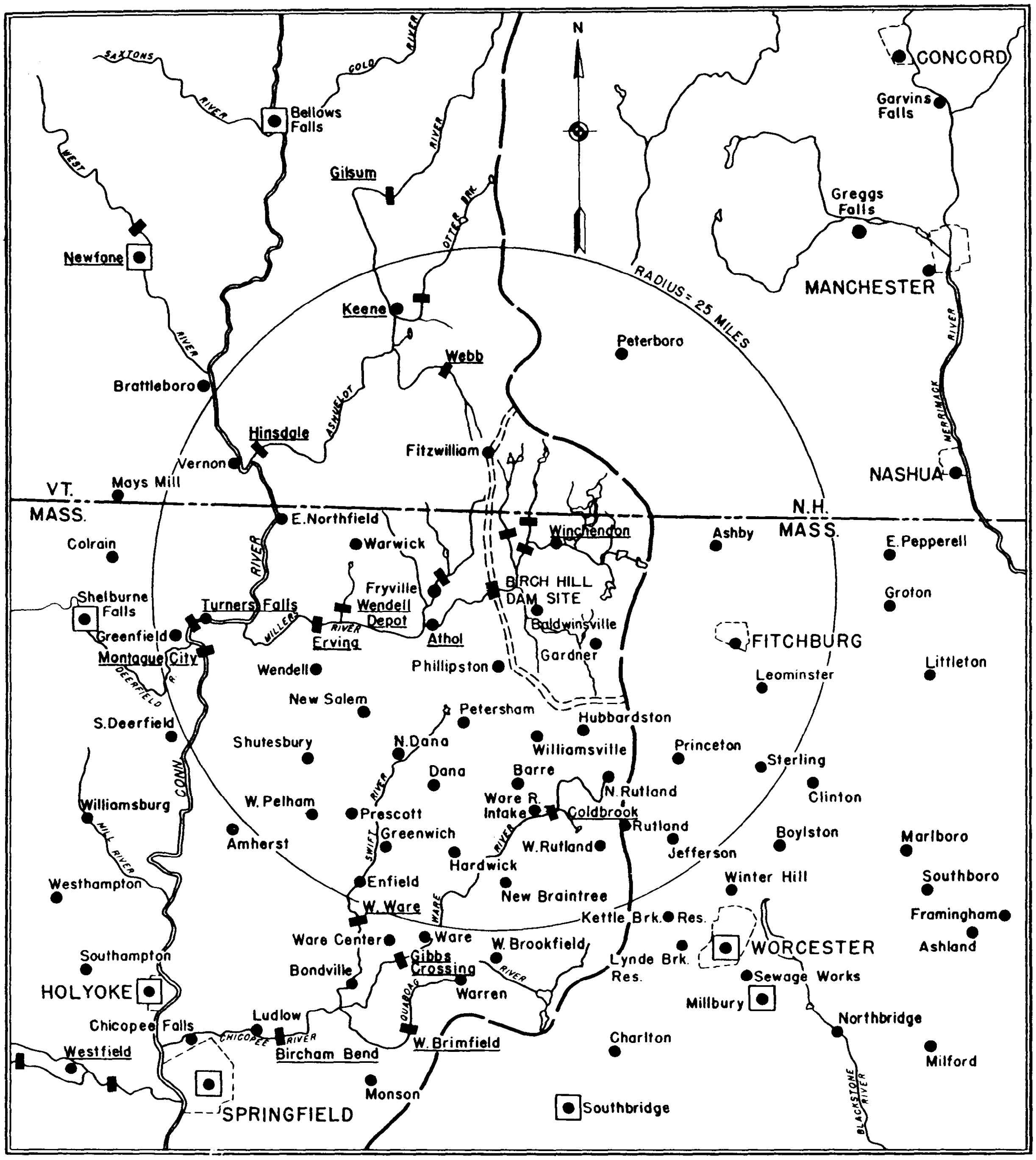


PLATE NO. 8

A. of D. G.49



LEGEND

- ● Rainfall Station
- ☐ Snow Cover Station
- ⊡ Rainfall And Snow Cover Station
- Stream Gaging Station
- Birch Hill Watershed
- Connecticut River Watershed

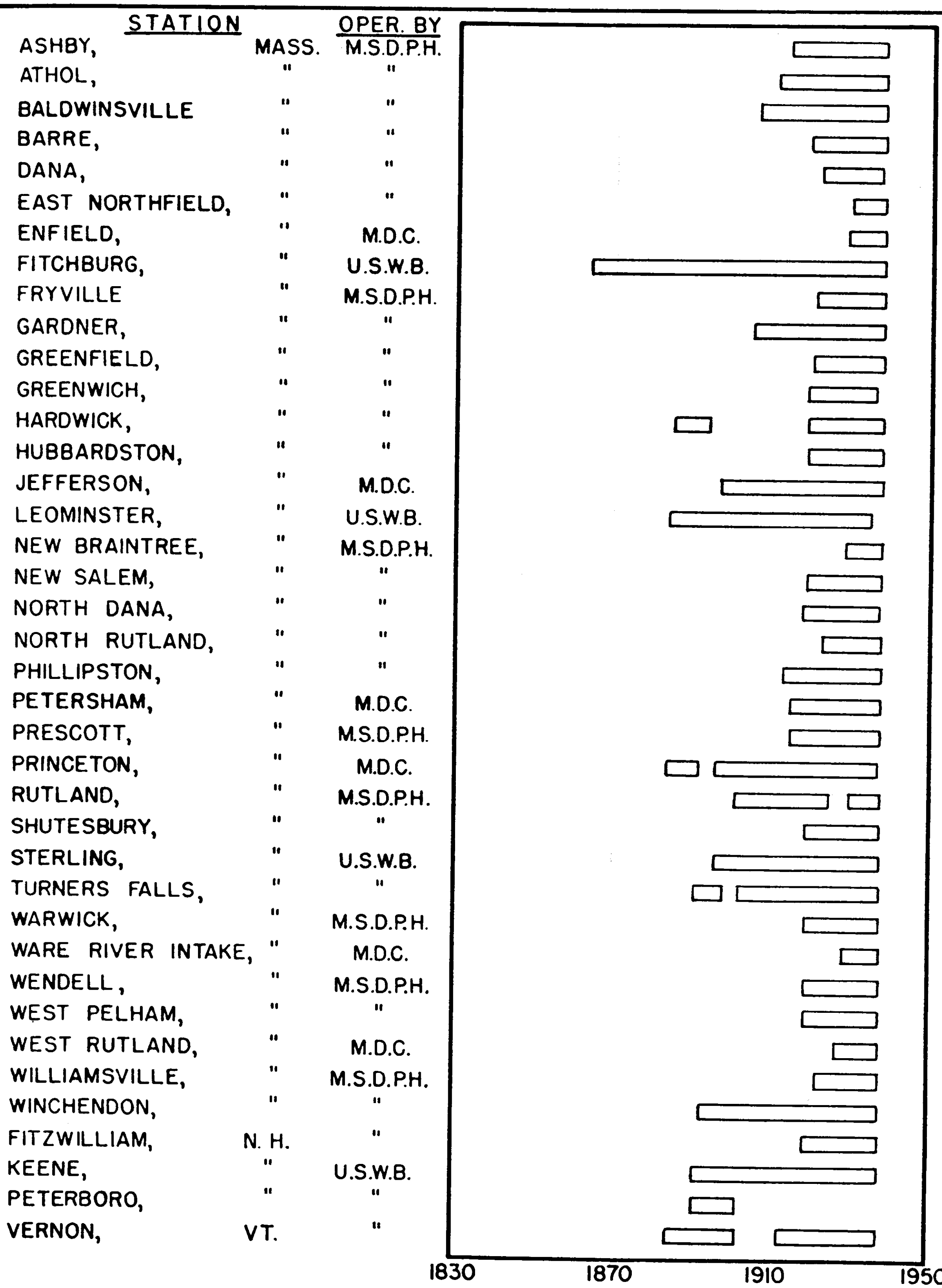
CONNECTICUT RIVER FLOOD CONTROL

RAINFALL AND STREAM GAGING STATIONS
IN VICINITY OF BIRCH HILL WATERSHED

U.S. ENGINEER OFFICE PROVIDENCE, R.I.

10 MI. 0 10 MI. 20 MI.

PLATE NO. 9



U.S.W.B. — United States Weather Bureau M.D.C. — Metropolitan District Commission
M.S.D.P.H — Massachusetts State Department of Public Health

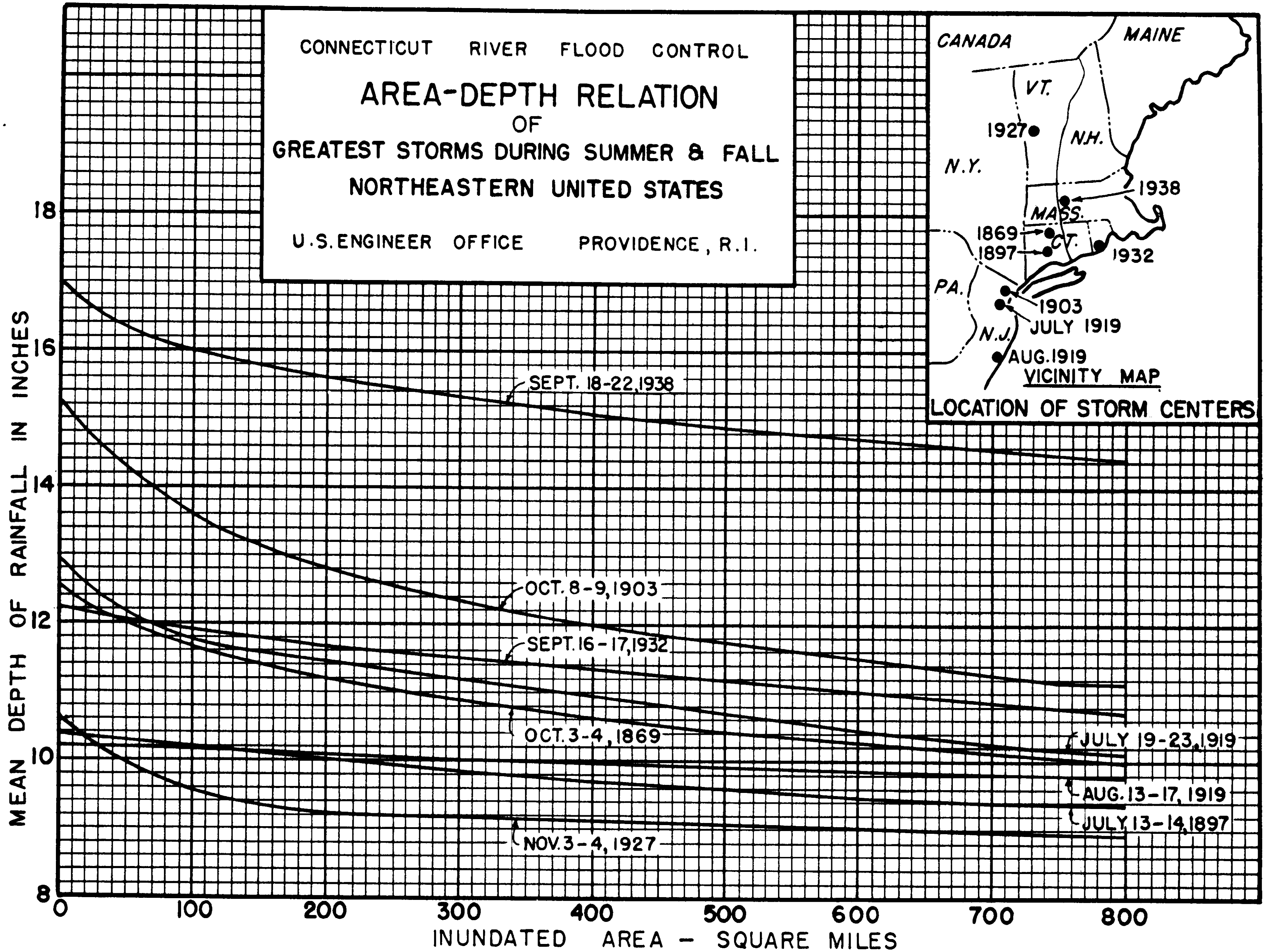
CONNECTICUT RIVER FLOOD CONTROL

YEARS OF RECORD OF RAINFALL STATIONS IN VICINITY OF BIRCH HILL WATERSHED

U. S. ENGINEER OFFICE

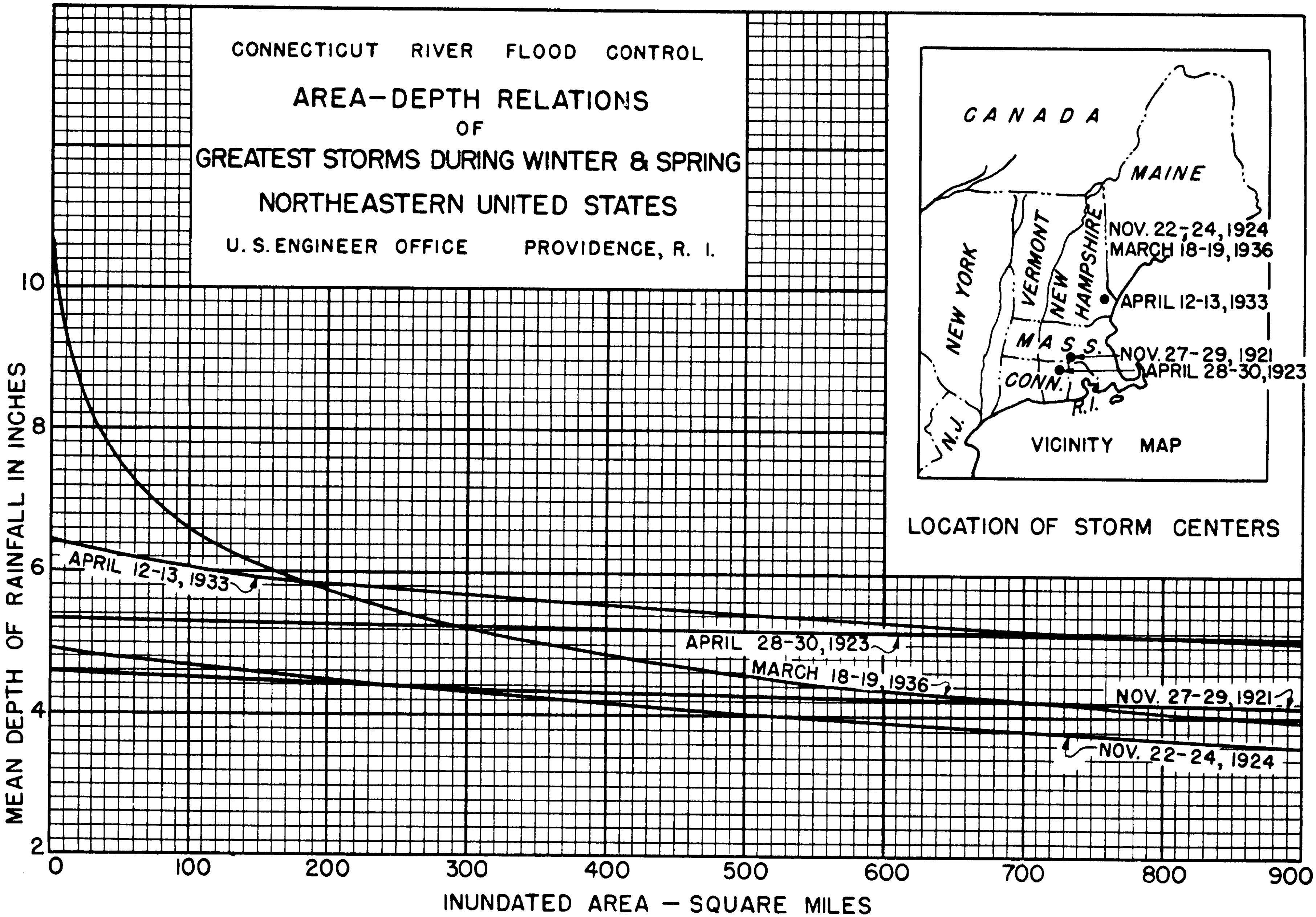
PROVIDENCE, R. I.

PLATE NO. 10



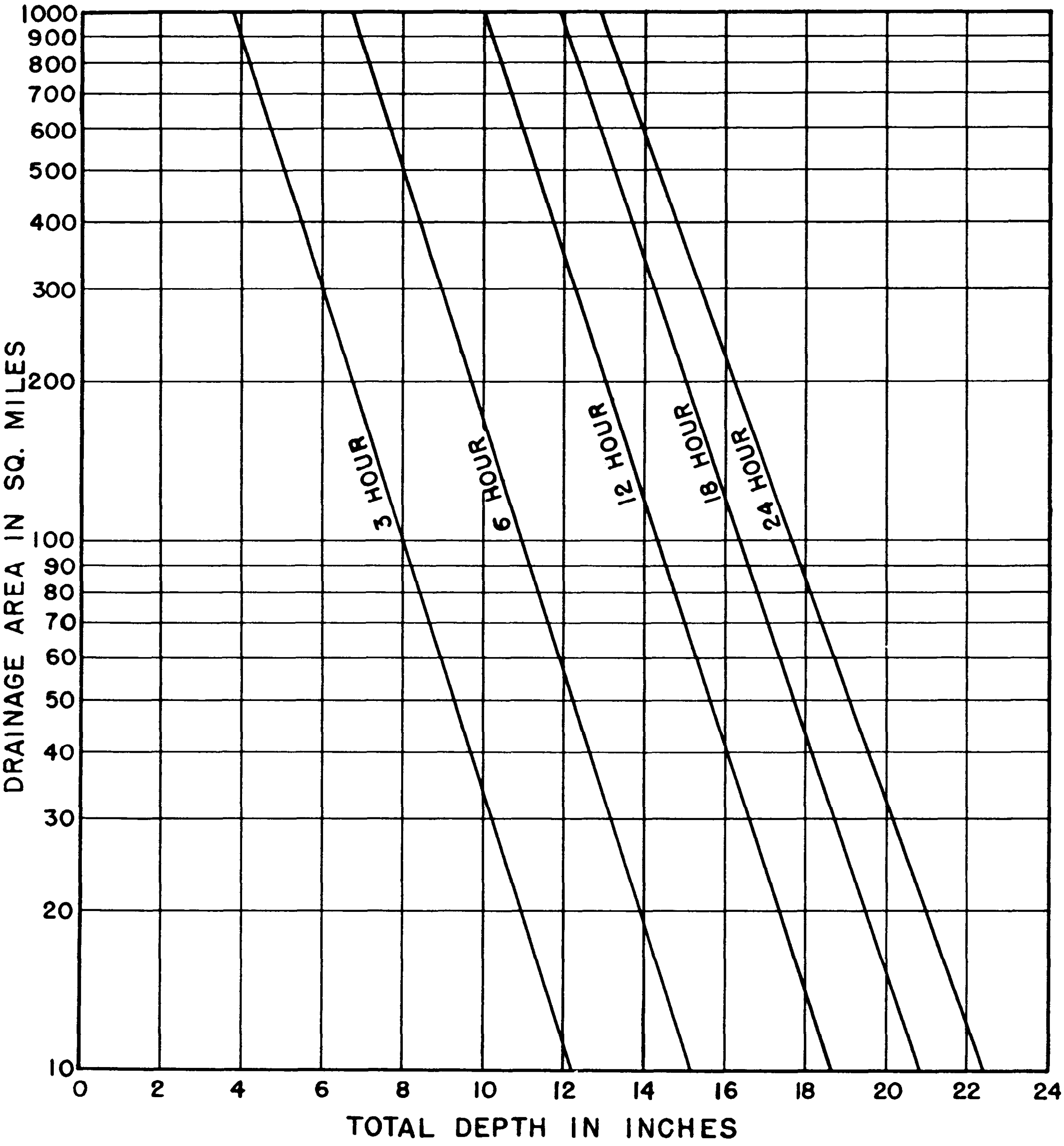
CONNECTICUT RIVER FLOOD CONTROL
AREA-DEPTH RELATION
OF
GREATEST STORMS DURING SUMMER & FALL
NORTHEASTERN UNITED STATES
U.S. ENGINEER OFFICE PROVIDENCE, R.I.
CANADA
MAINE
VT.
N.H.
N.Y.
MASS.
CT.
PA.
N.J.
1927
1938
1869
1897
1932
1903
JULY 1919
AUG. 1919
VICINITY MAP
LOCATION OF STORM CENTERS
SEPT. 18-22, 1938
OCT. 8-9, 1903
SEPT. 16-17, 1932
OCT. 3-4, 1869
JULY 19-23, 1919
AUG. 13-17, 1919
JULY 13-14, 1897
NOV. 3-4, 1927
MEAN DEPTH OF RAINFALL IN INCHES
8
10
12
14
16
18
0
100
200
300
400
500
600
700
800
INUNDATED AREA - SQUARE MILES

PLATE NO. 11



CONNECTICUT RIVER FLOOD CONTROL
AREA-DEPTH RELATIONS
OF
GREATEST STORMS DURING WINTER & SPRING
NORTHEASTERN UNITED STATES
U. S. ENGINEER OFFICE PROVIDENCE, R. I.
CANADA
MAINE
NEW YORK
VERMONT
NEW HAMPSHIRE
MASS.
CONN.
R.I.
N.J.
NOV. 22-24, 1924
MARCH 18-19, 1936
APRIL 12-13, 1933
NOV. 27-29, 1921
APRIL 28-30, 1923
VICINITY MAP
LOCATION OF STORM CENTERS
APRIL 12-13, 1933
APRIL 28-30, 1923
MARCH 18-19, 1936
NOV. 27-29, 1921
NOV. 22-24, 1924
MEAN DEPTH OF RAINFALL IN INCHES
2
4
6
8
10
0
100
200
300
400
500
600
700
800
900
INUNDATED AREA — SQUARE MILES

PLATE NO. 12



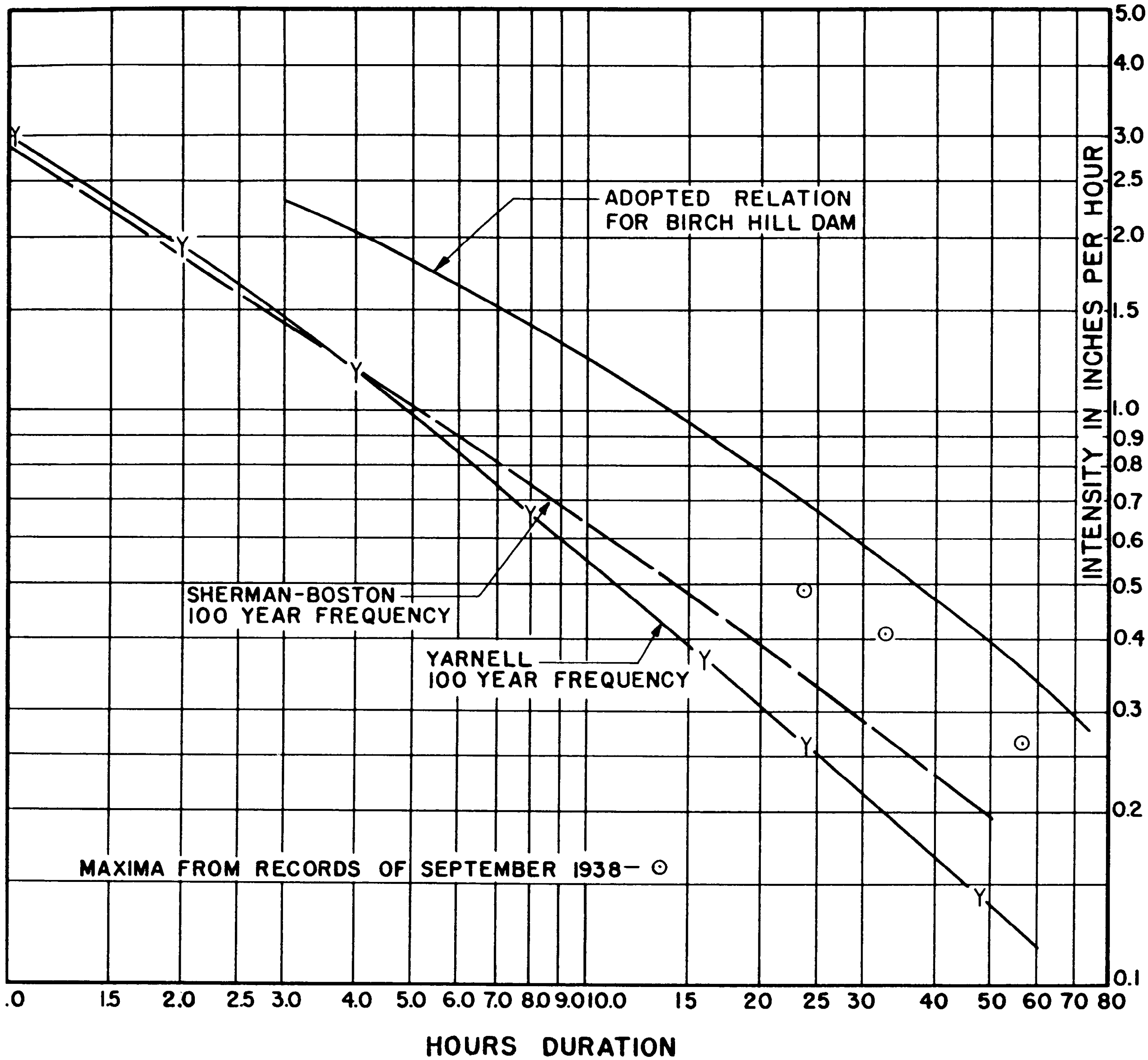
CONNECTICUT RIVER FLOOD CONTROL

BIRCH HILL DAM

AREA-DEPTH-DURATION RELATION
FOR NORTHEASTERN UNITED STATES

U.S ENGINEER OFFICE, PROVIDENCE, R.I.

PLATE NO. 13



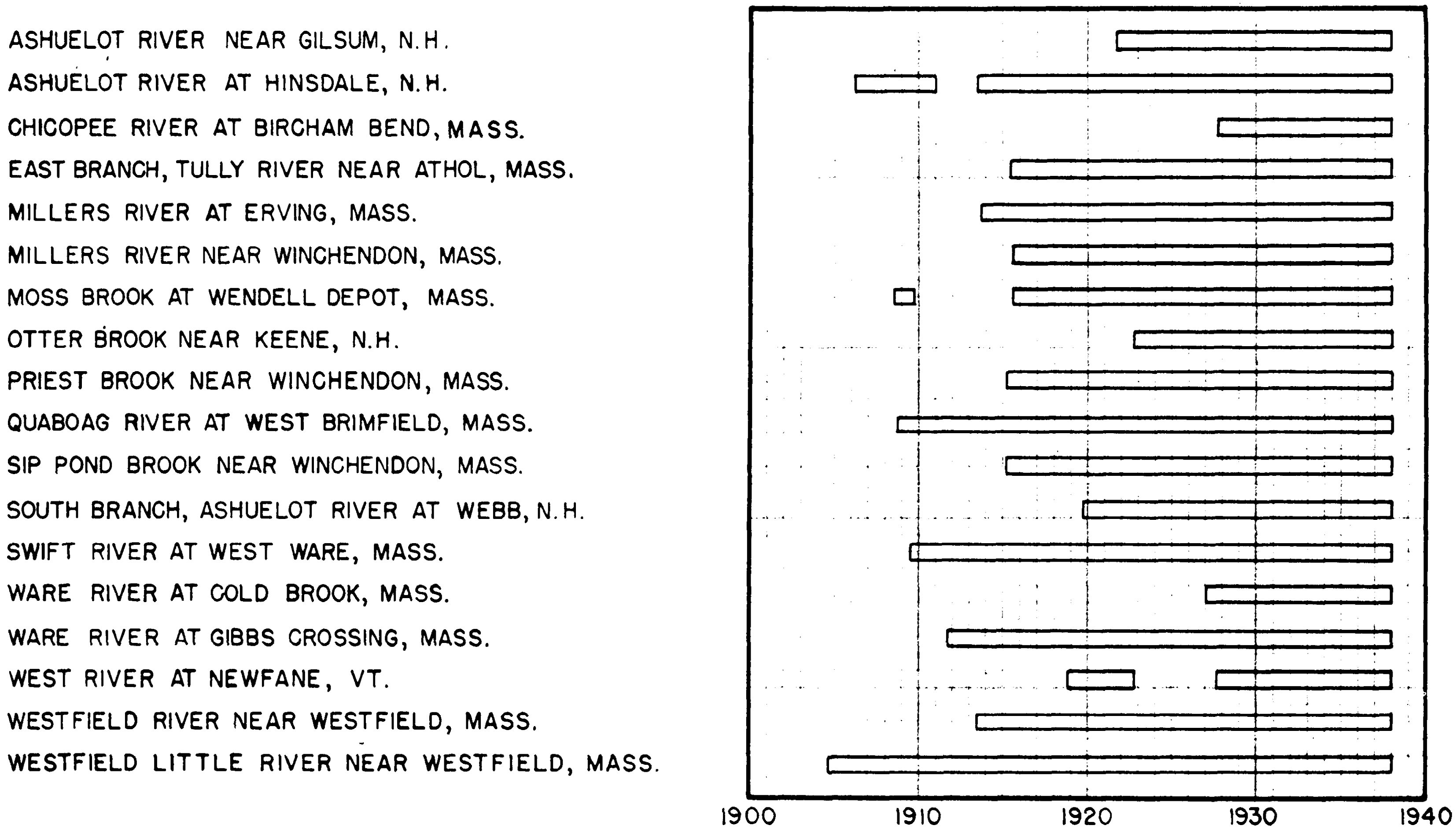
CONNECTICUT RIVER FLOOD CONTROL

BIRCH HILL DAM

MAXIMUM RAINFALL INTENSITY VERSUS DURATION

U.S. ENGINEER OFFICE PROVIDENCE, R.I.

PLATE NO. 14

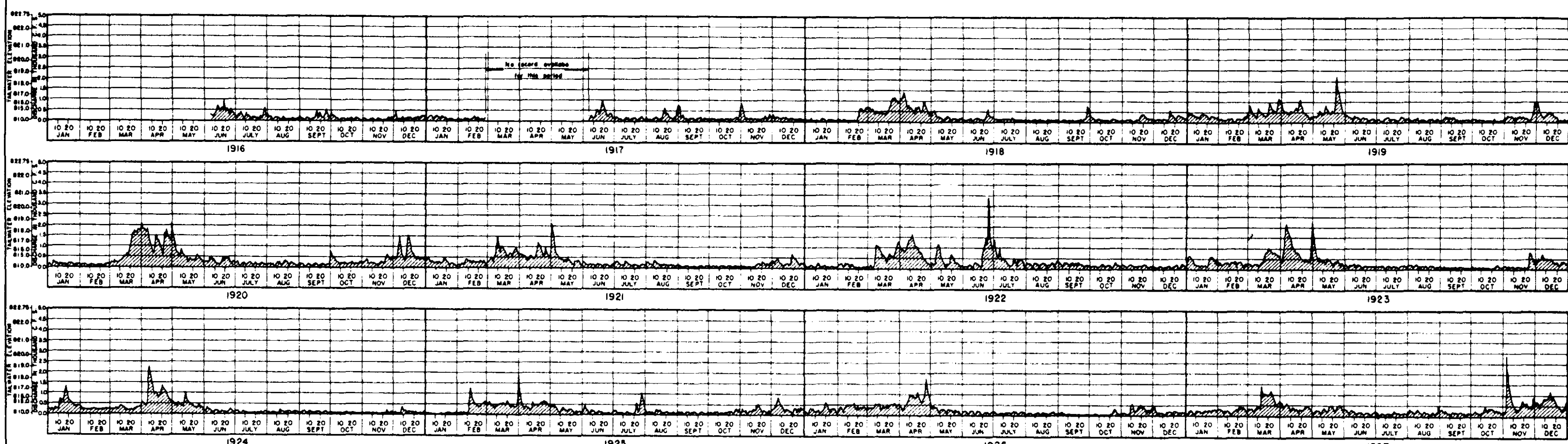


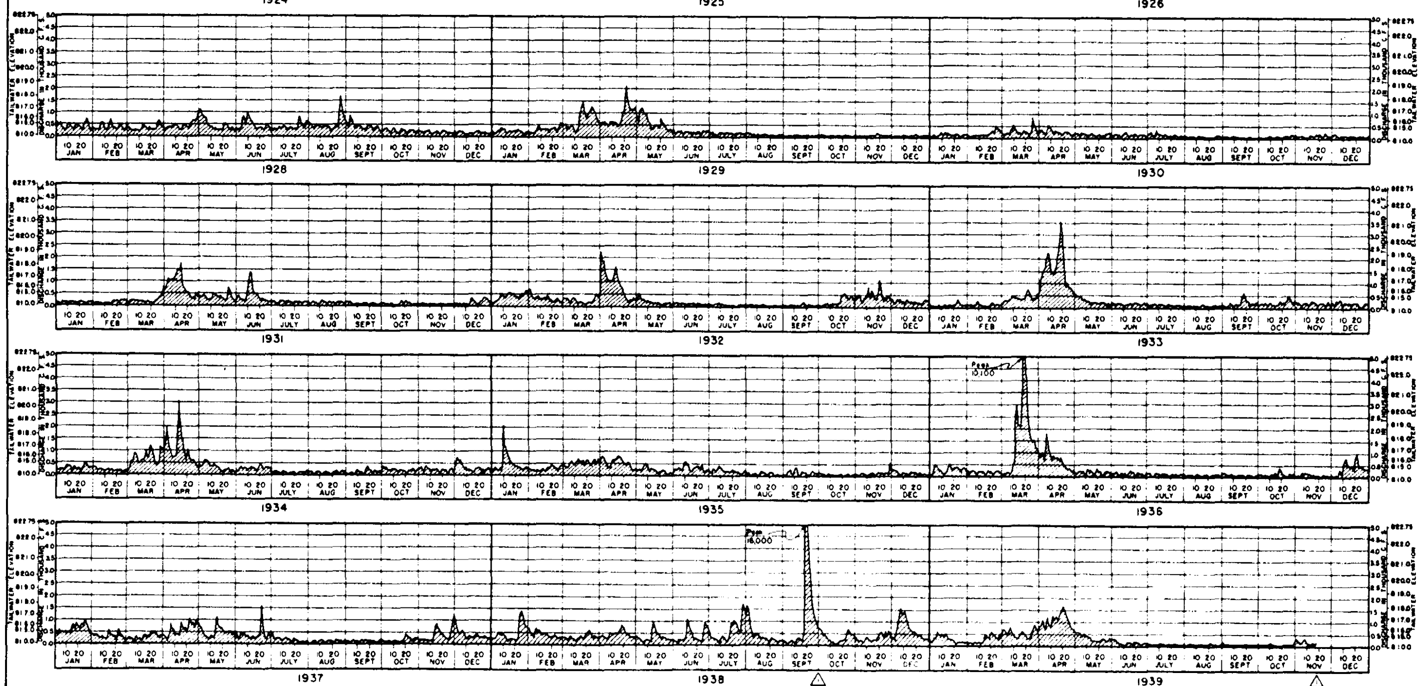
CONNECTICUT RIVER FLOOD CONTROL

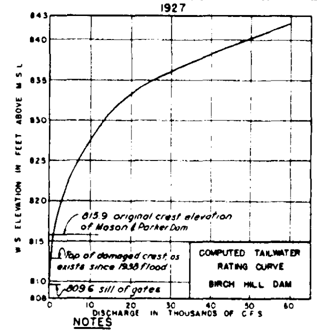
YEARS OF RECORD OF STREAM GAGING STATIONS
IN VICINITY OF BIRCH HILL WATERSHED

U.S. ENGINEER OFFICE — PROVIDENCE, R.I.

PLATE NO. 15







NOTES

These hydrographs obtained by using the records of the U.S.G.S. Gaging Station on the Millers River at Winchendon, Mass (D.A. 83.8 Sq. Mi.) and applying the factors shown in the following table. Drainage area at damsite is 175 sq. miles

Flow at Winchendon	Factor
Less than 700 c.f.s.	2.1
From 700 to 1100 c.f.s.	2.0
More than 1100 c.f.s.	1.9

The plotted values are average daily discharges except for peak flows above 1800 c.f.s. which are instantaneous. Elevations are in feet above Mean Sea Level.

KEY	DATE	REVISION (indicated by △)	REV BY	CK BY	AP BY
△	2-5-40	Addition to Hydrograph Oct. 1, 1930 to Nov. 17, 1939.			

CONNECTICUT RIVER FLOOD CONTROL

BIRCH HILL DAM

HYDROGRAPHS

MILLERS RIVER — MASSACHUSETTS

IN 49 SHEETS — SCALE AS SHOWN — SHEET NO 2

U. S. ENGINEER OFFICE, PROVIDENCE, R. I. — JAN., 1940

SUBMITTED: John B. Drake, ENGINEER, HEAD, HYDRAULICS SECTION

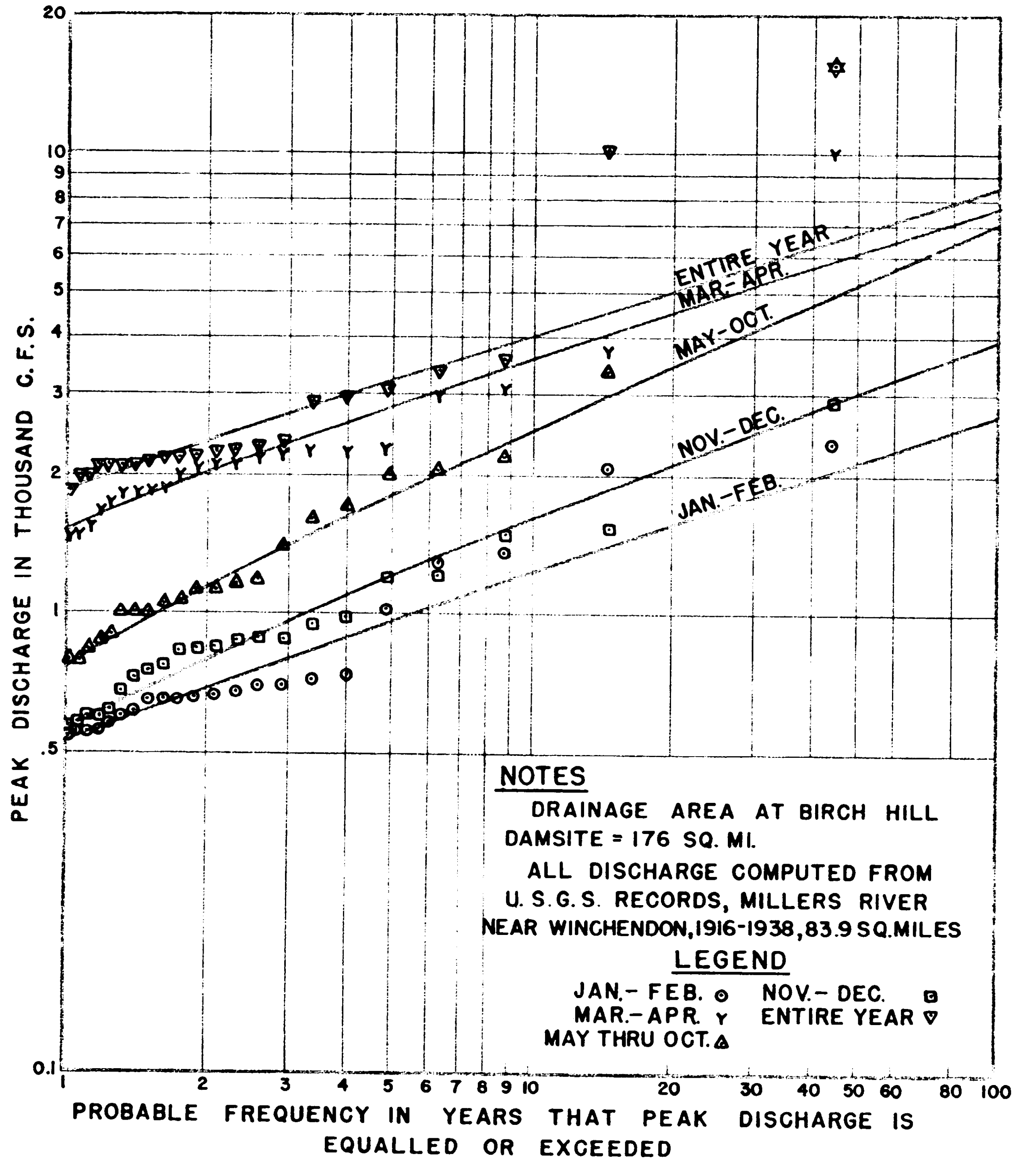
APPROVAL RECOMMENDED: J. S. Burns, PRINCIPAL ENGINEER, CHIEF, F. C. ENGINEERING DIV.

APPROVED: J. Teagarden, LT. COL., CORPS OF ENGINEERS, DISTRICT ENGINEER

COMPILED: George H. Whitehead, ASSOC. ENGINEER

DRAWN: HHG & ALR
TRACED: B.A.J.
CHECKED: J.C.W.

FILE NO. CT-3-1097



CONNECTICUT RIVER FLOOD CONTROL

ESTIMATED SEASONAL FREQUENCY OF PEAK DISCHARGE

BIRCH HILL DAMSITE

U. S. ENGINEER OFFICE

PROVIDENCE, R. I.

PLATE NO. 17

CONNECTICUT RIVER FLOOD CONTROL

BIRCH HILL DAM

MAXIMUM PREDICTED FLOOD

U.S. ENGINEER OFFICE, PROVIDENCE, R.I.

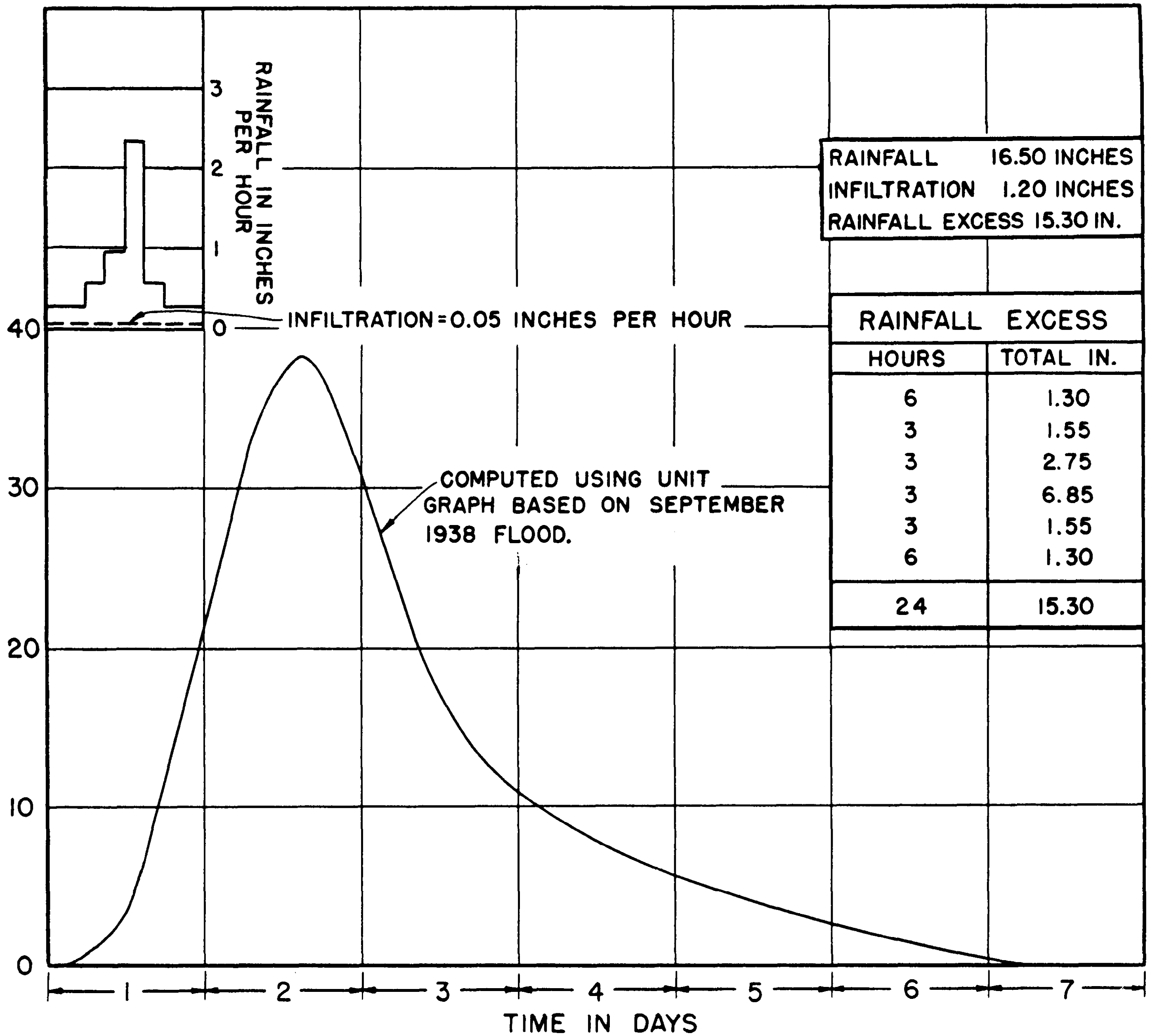
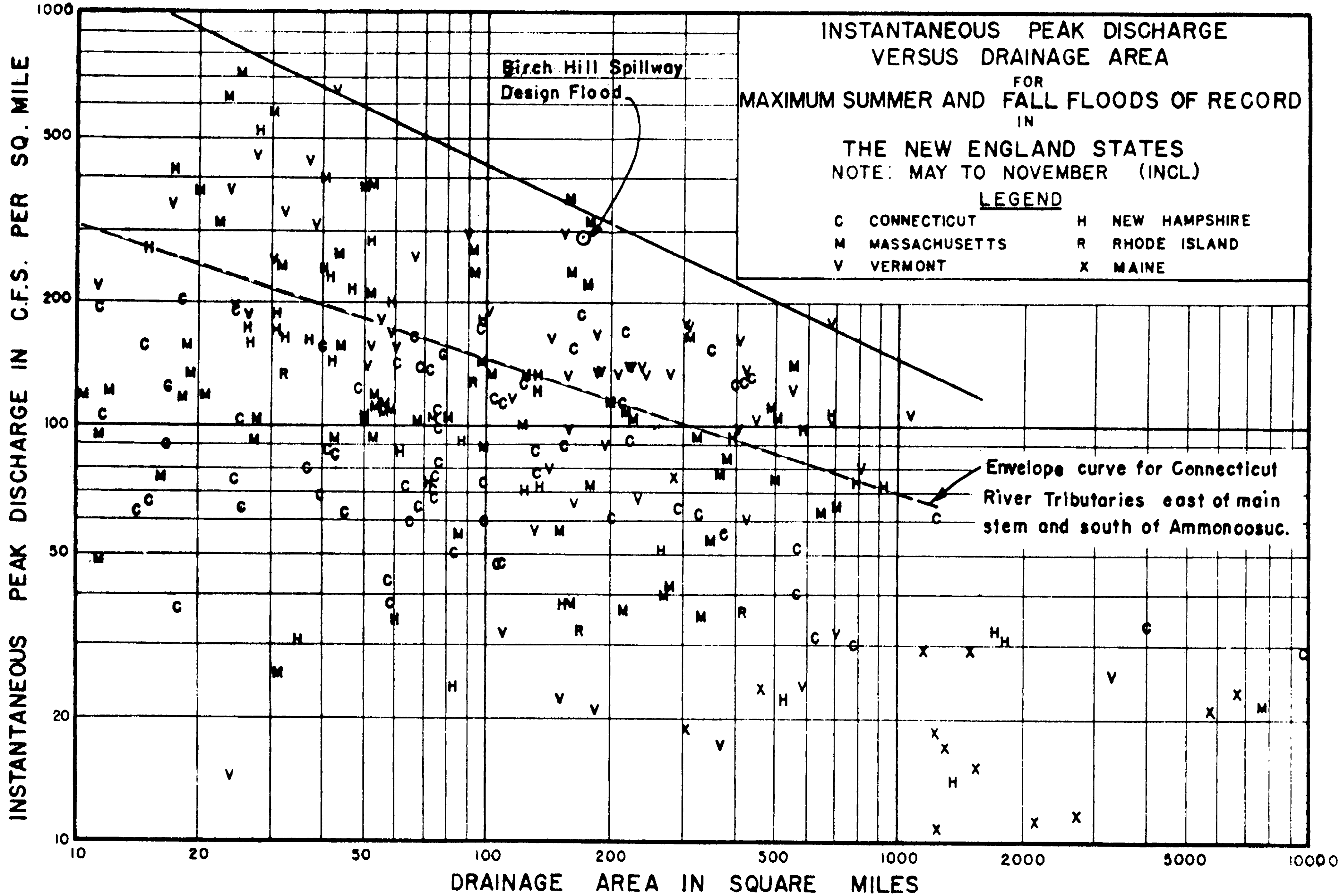


PLATE NO. 18



INSTANTANEOUS PEAK DISCHARGE
VERSUS DRAINAGE AREA
FOR
MAXIMUM SUMMER AND FALL FLOODS OF RECORD
IN
THE NEW ENGLAND STATES
NOTE: MAY TO NOVEMBER (INCL)
LEGEND
C CONNECTICUT
M MASSACHUSETTS
V VERMONT
H NEW HAMPSHIRE
R RHODE ISLAND
X MAINE
Birch Hill Spillway Design Flood
Envelope curve for Connecticut River Tributaries east of main stem and south of Ammonoosuc.
INSTANTANEOUS PEAK DISCHARGE IN C.F.S. PER SQ. MILE
1000
500
200
100
50
20
10
10
20
50
100
200
500
1000
2000
5000
10000
DRAINAGE AREA IN SQUARE MILES

PLATE NO. 19

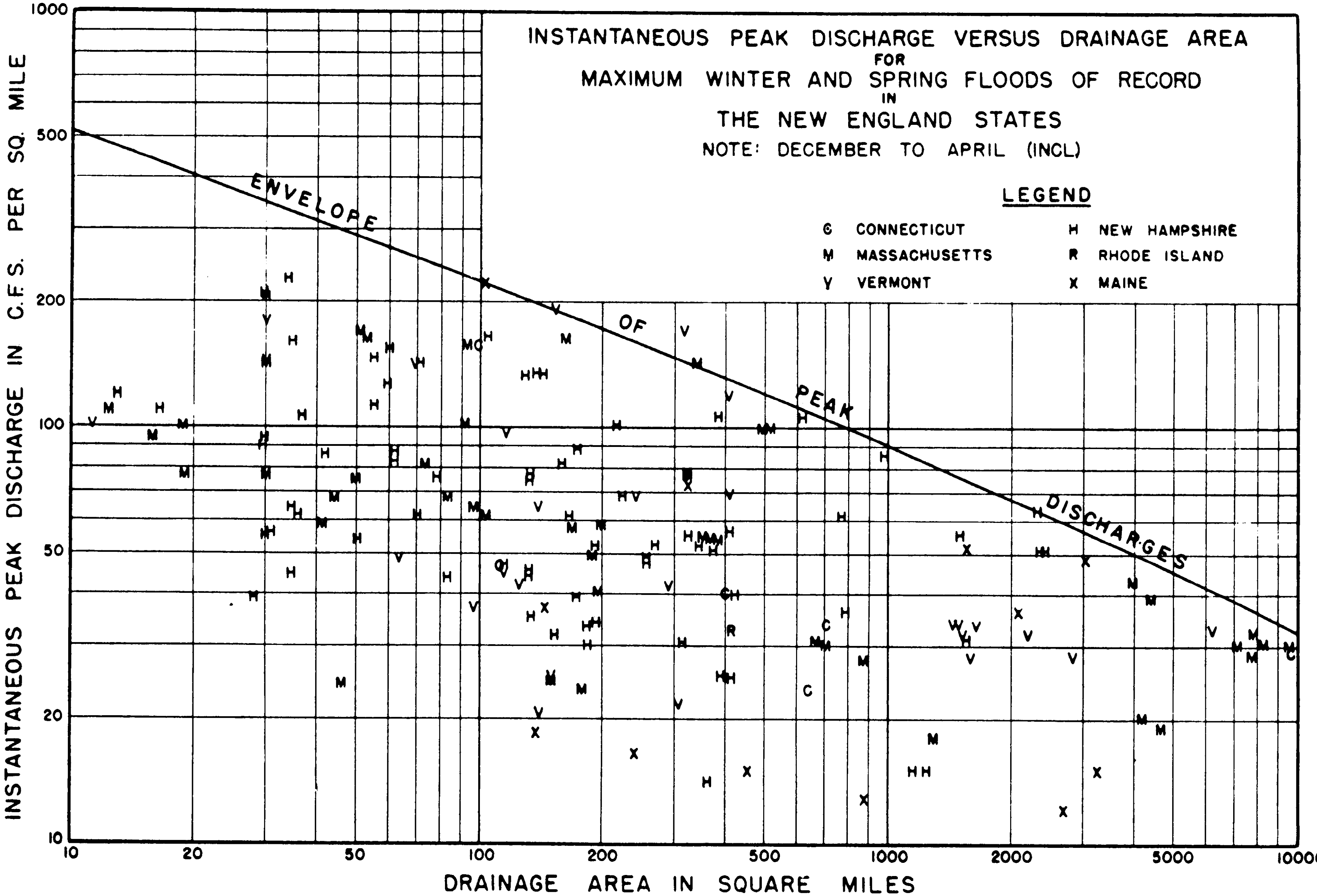
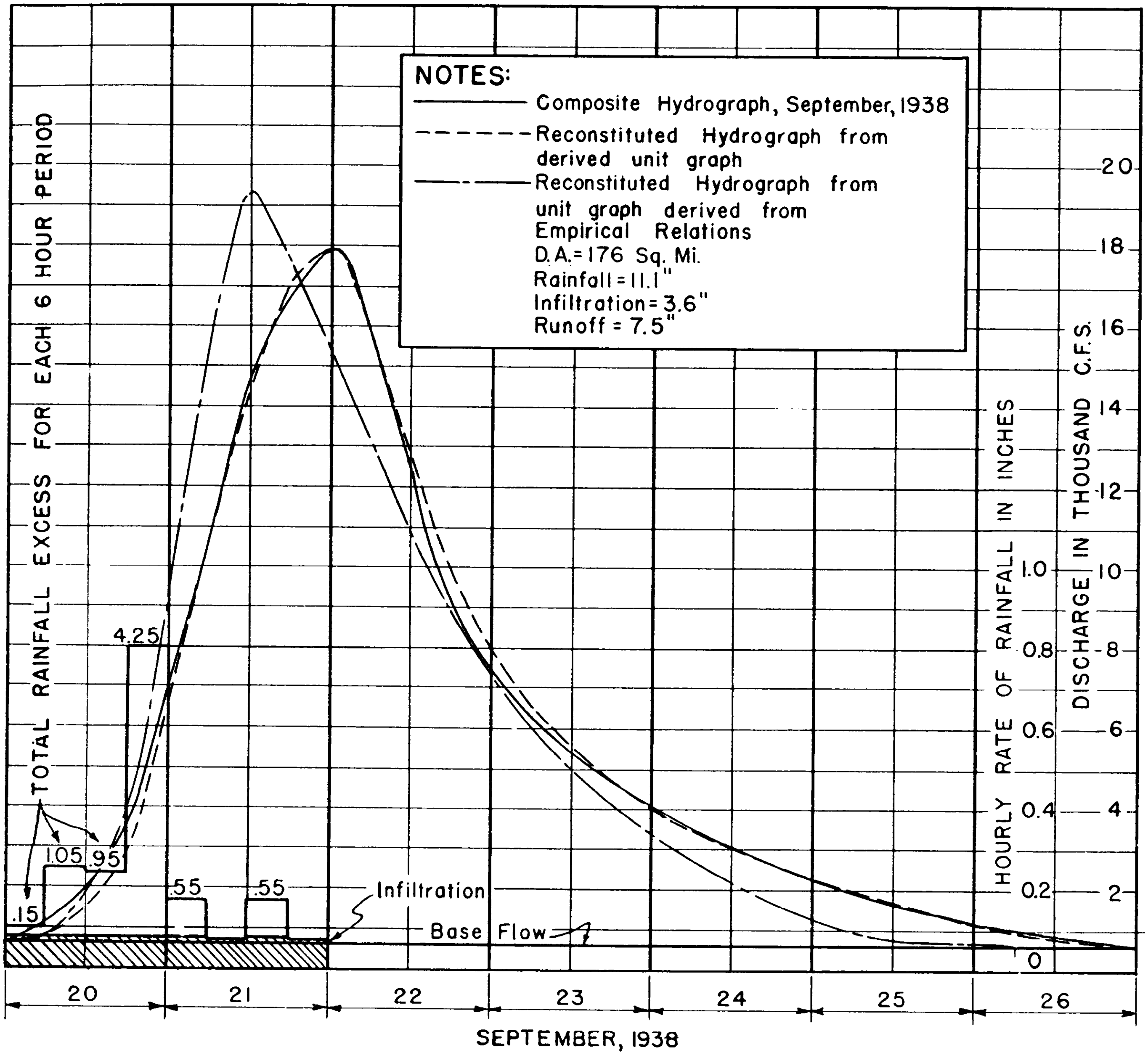

INSTANTANEOUS PEAK DISCHARGE VERSUS DRAINAGE AREA
FOR
MAXIMUM WINTER AND SPRING FLOODS OF RECORD
IN
THE NEW ENGLAND STATES
NOTE: DECEMBER TO APRIL (INCL)
LEGEND
C CONNECTICUT
M MASSACHUSETTS
V VERMONT
H NEW HAMPSHIRE
R RHODE ISLAND
X MAINE
ENVELOPE OF PEAK DISCHARGES
INSTANTANEOUS PEAK DISCHARGE IN C.F.S. PER SQ. MILE
DRAINAGE AREA IN SQUARE MILES
1000
500
200
100
50
20
10
10
20
50
100
200
500
1000
2000
5000
10000

PLATE NO. 20



CONNECTICUT RIVER FLOOD CONTROL

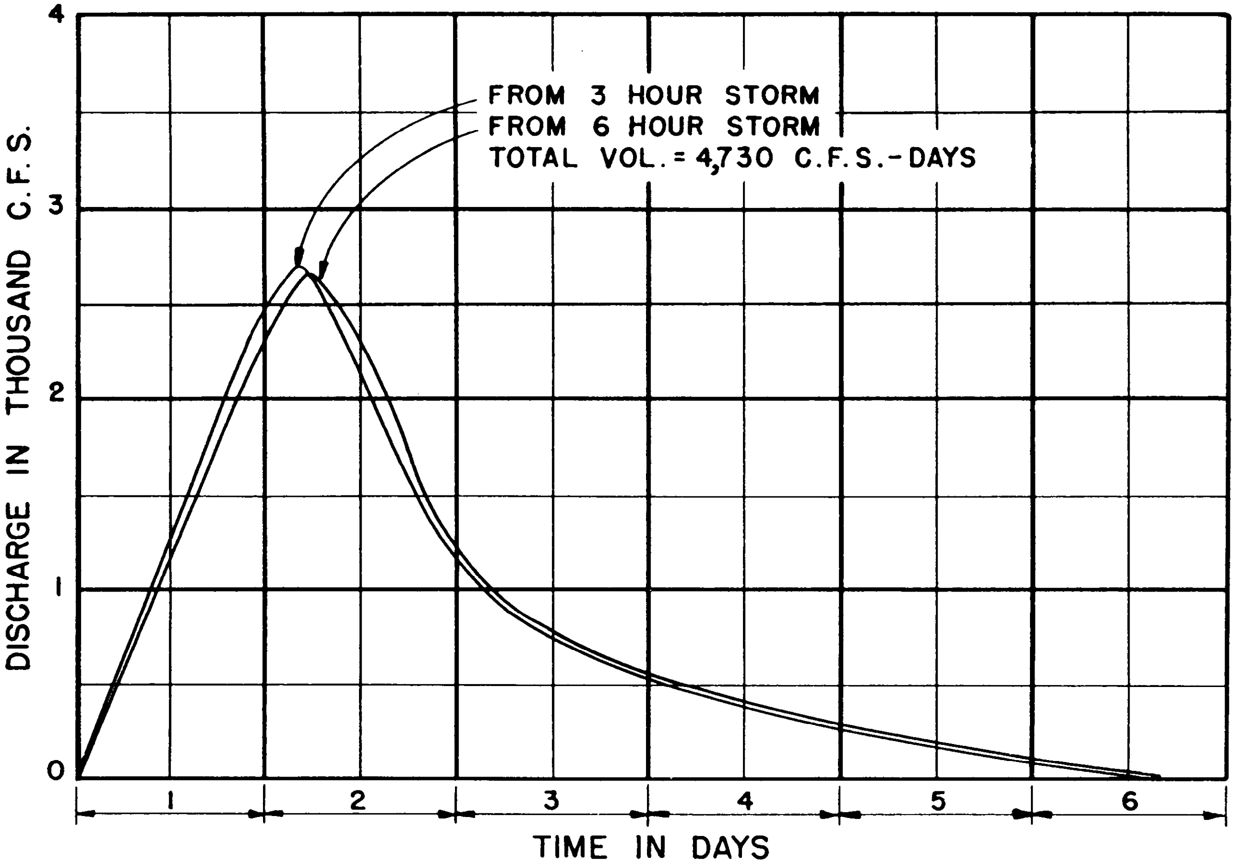
DERIVATION OF UNIT HYDROGRAPH

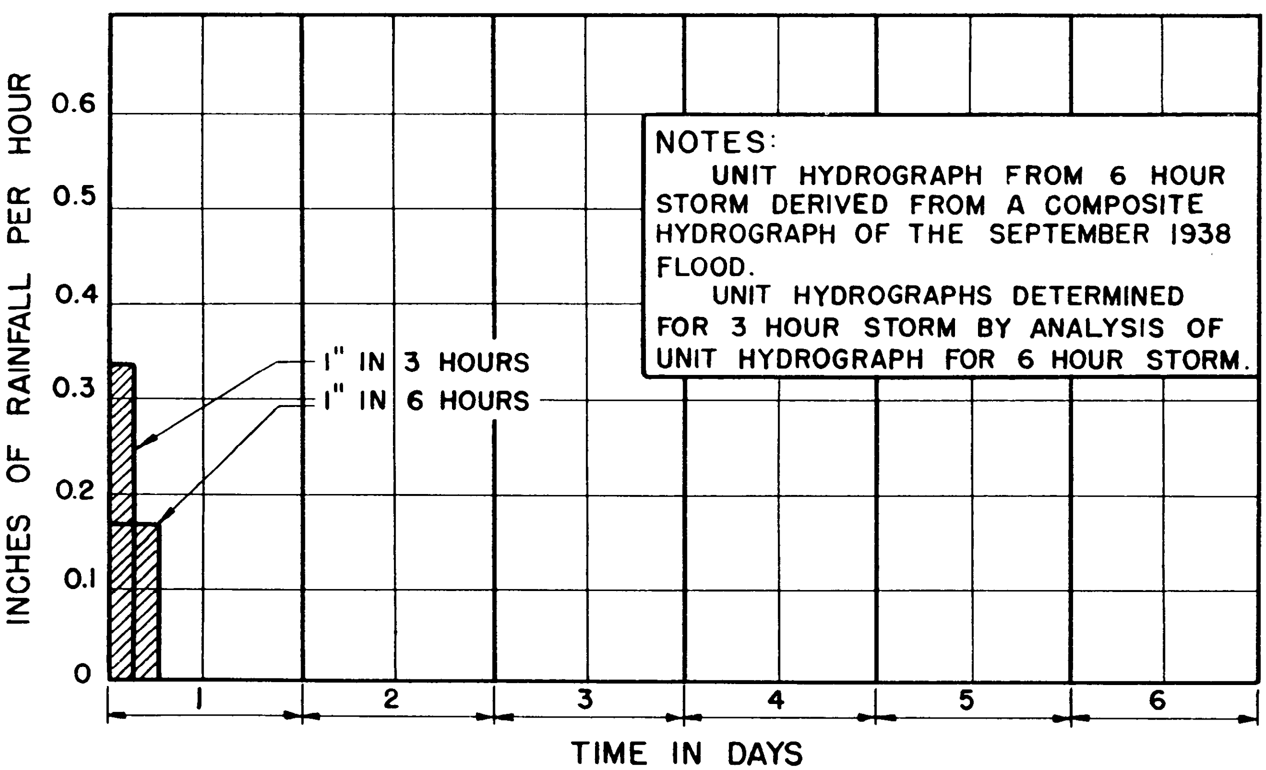
SEPTEMBER 1938 FLOOD

BIRCH HILL DAM SITE

U.S. ENGINEER OFFICE PROVIDENCE, R.I.

PLATE NO. 21





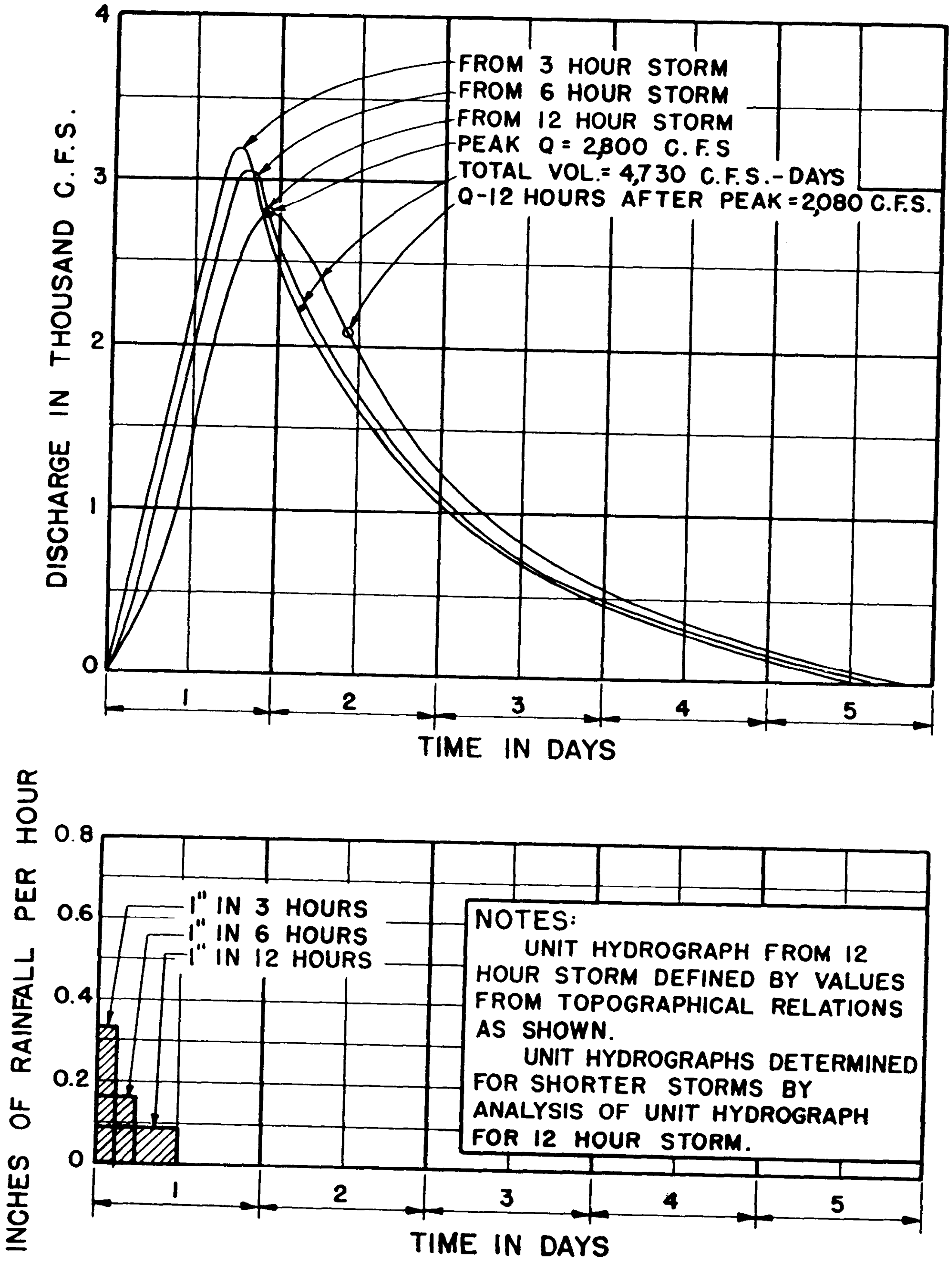
CONNECTICUT RIVER FLOOD CONTROL

BIRCH HILL DAM

UNIT HYDROGRAPHS DERIVED FROM SEPTEMBER 1938 FLOOD

U. S. ENGINEER OFFICE PROVIDENCE, R. I.

PLATE NO. 22



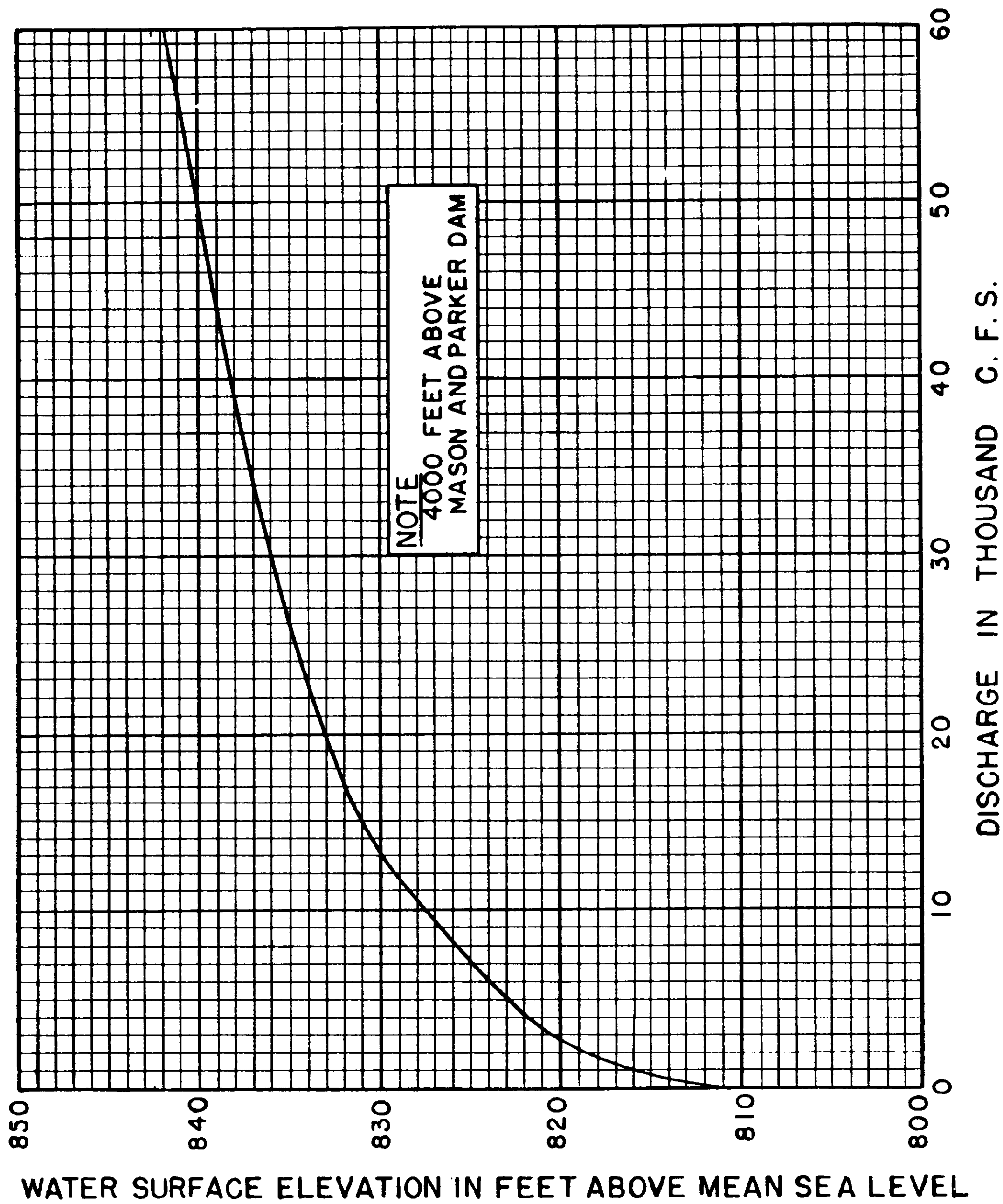
CONNECTICUT RIVER FLOOD CONTROL

BIRCH HILL DAM

UNIT HYDROGRAPHS DERIVED FROM EMPIRICAL RELATIONS

U.S. ENGINEER OFFICE, PROVIDENCE, R. I.

PLATE NO. 23



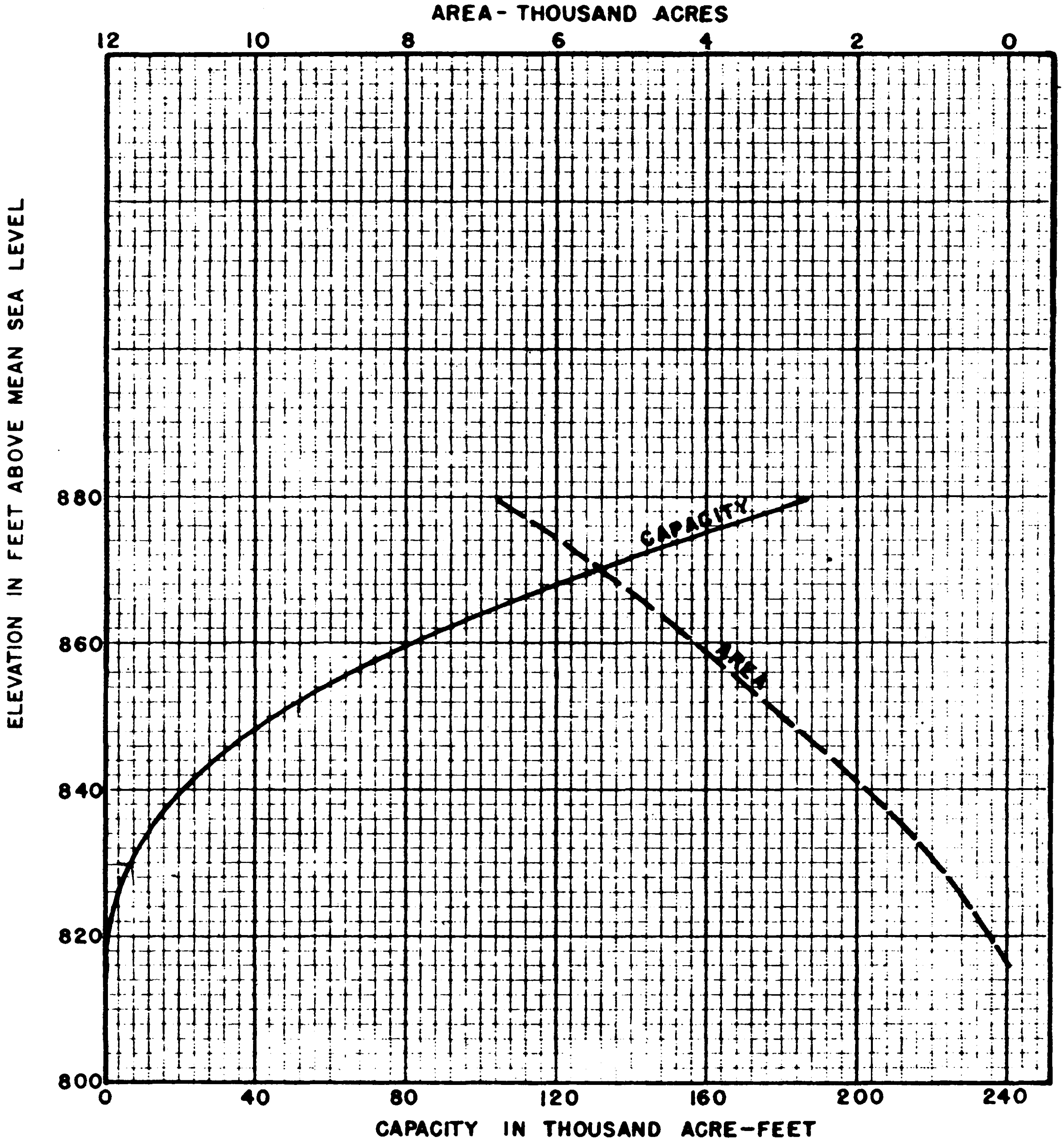
CONNECTICUT RIVER FLOOD CONTROL

BIRCH HILL DAM

TAILWATER RATING CURVE

U.S. ENGINEER OFFICE, PROVIDENCE, R.I.

PLATE NO. 24



SOURCE OF DATA U.S.E.D. RESERVOIR SURVEY OF OCT. 1939

CONNECTICUT RIVER FLOOD CONTROL

BIRCH HILL RESERVOIR

MILLERS RIVER MASSACHUSETTS

U.S. ENGINEER OFFICE PROVIDENCE, R.I.

JAN. 1940

PLATE NO. 25

CONNECTICUT RIVER FLOOD CONTROL

BIRCH HILL DAM

OUTLET DESIGN FLOOD

U.S. ENGINEER OFFICE, PROVIDENCE, R.I.

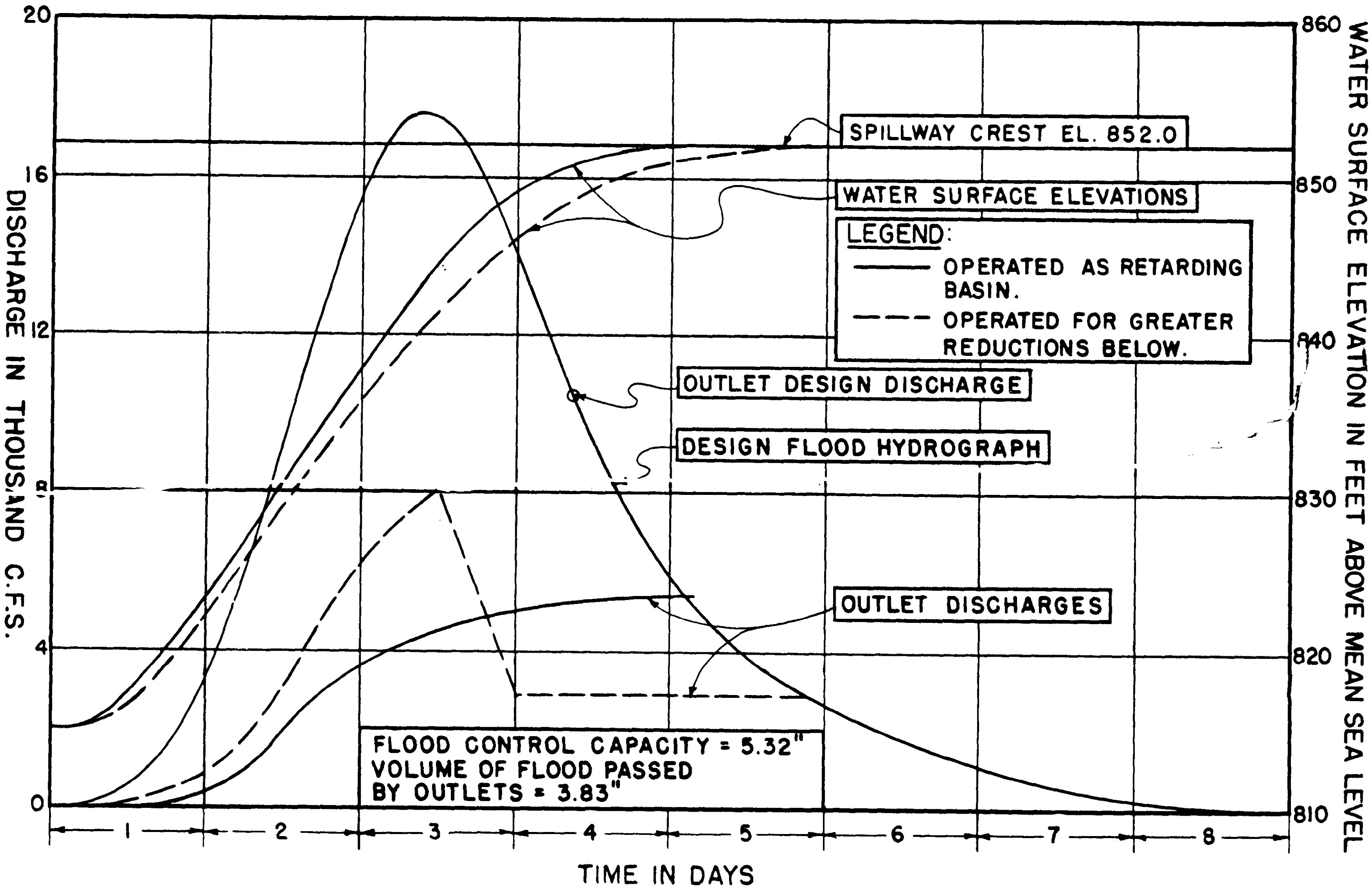
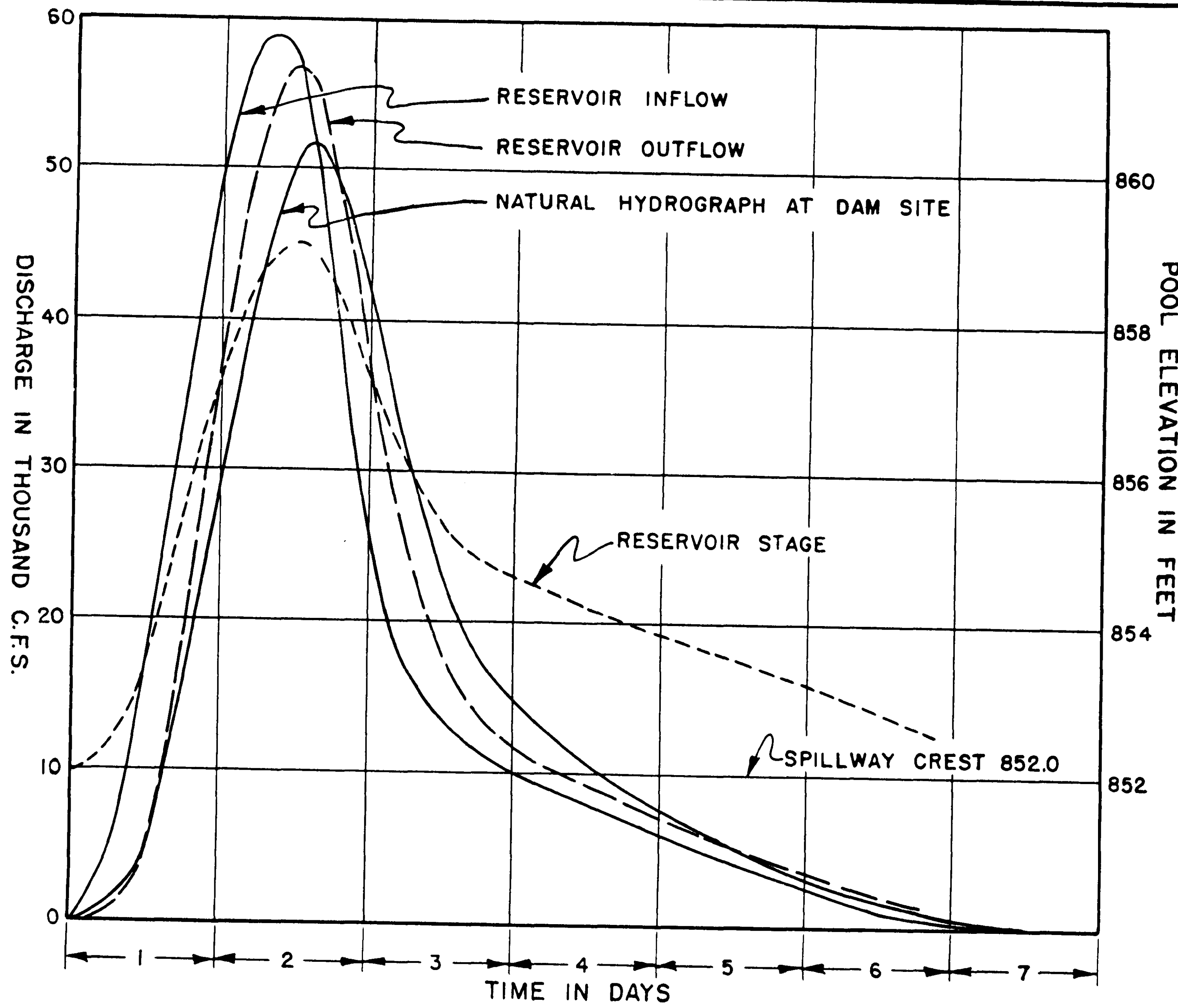


PLATE NO. 26



CONNECTICUT RIVER FLOOD CONTROL

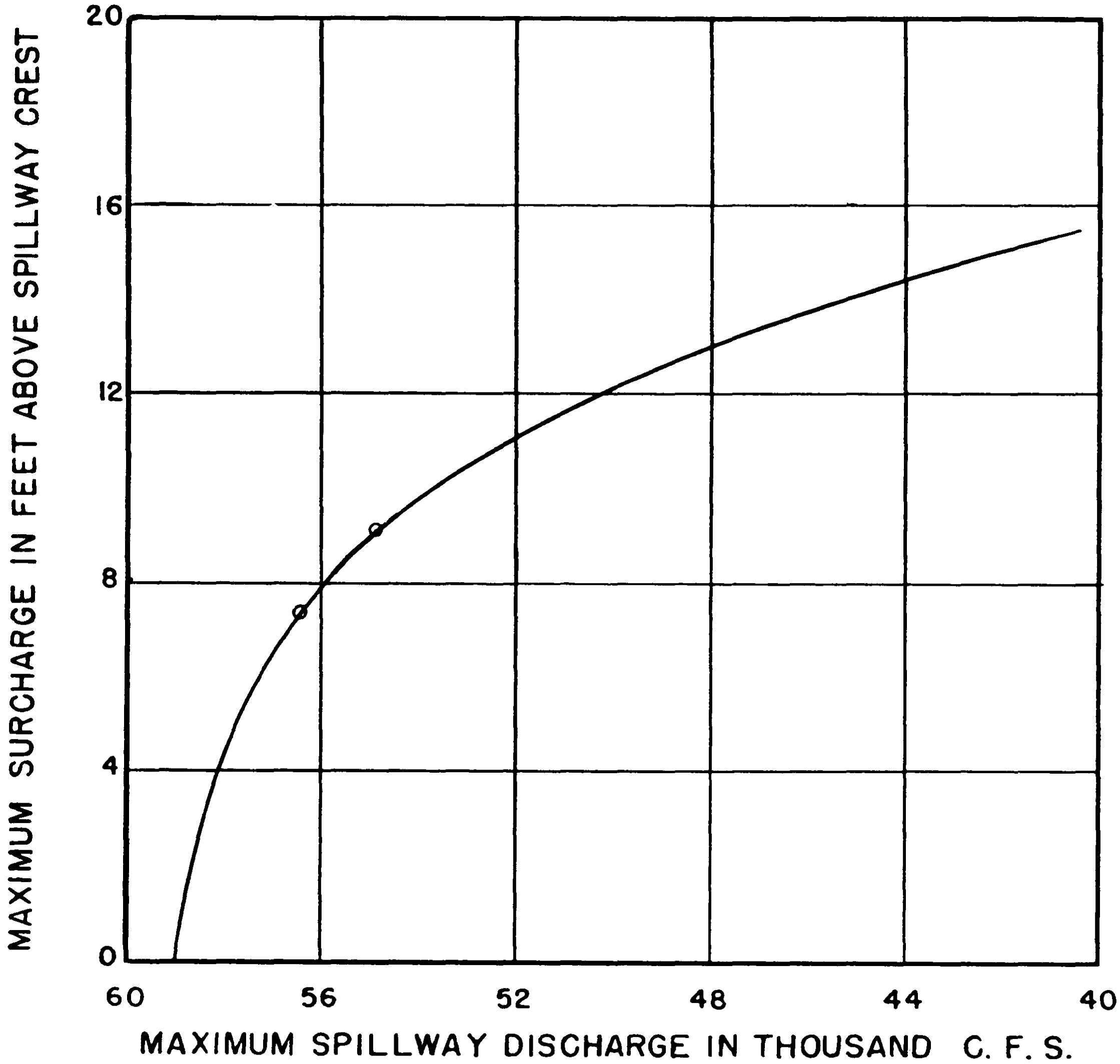
BIRCH HILL DAM

SPILLWAY DESIGN FLOOD

INFLOW, OUTFLOW, AND STAGE HYDROGRAPHS

U.S ENGINEER OFFICE, PROVIDENCE, R.I.

PLATE NO. 27

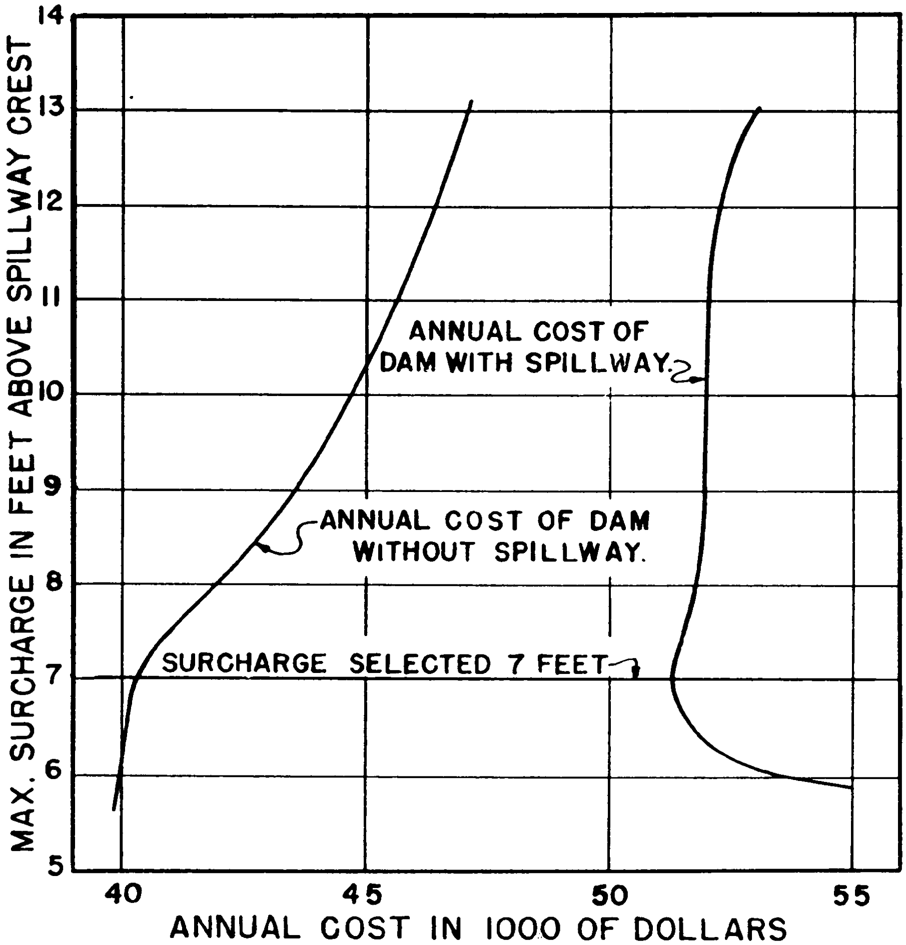


CONNECTICUT RIVER FLOOD CONTROL

BIRCH HILL DAM

MAXIMUM SURCHARGE - DISCHARGE

U.S. ENGINEER OFFICE, PROVIDENCE, R. I



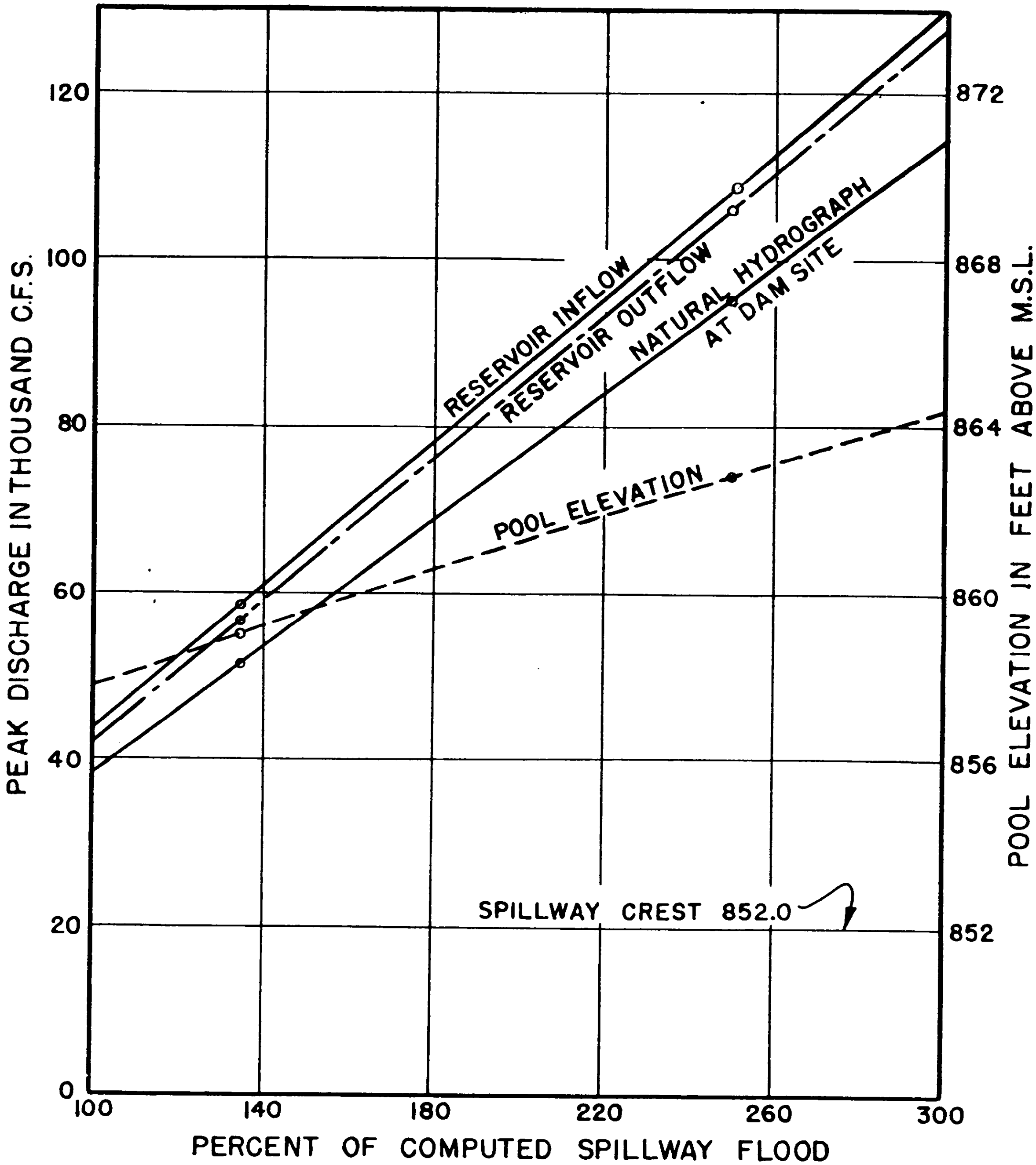
CONNECTICUT RIVER FLOOD CONTROL

BIRCH HILL DAM

ANNUAL COST VS. SURCHARGE

U.S. ENGINEER OFFICE, PROVIDENCE, R.I.

PLATE NO. 29



CONNECTICUT RIVER FLOOD CONTROL

BIRCH HILL DAM

OVERLOAD CHARACTERISTICS

U.S. ENGINEER OFFICE, PROVIDENCE, R.I.

PLATE NO. 30

PROVIDENCE DISTRICT SOIL CLASSIFICATION

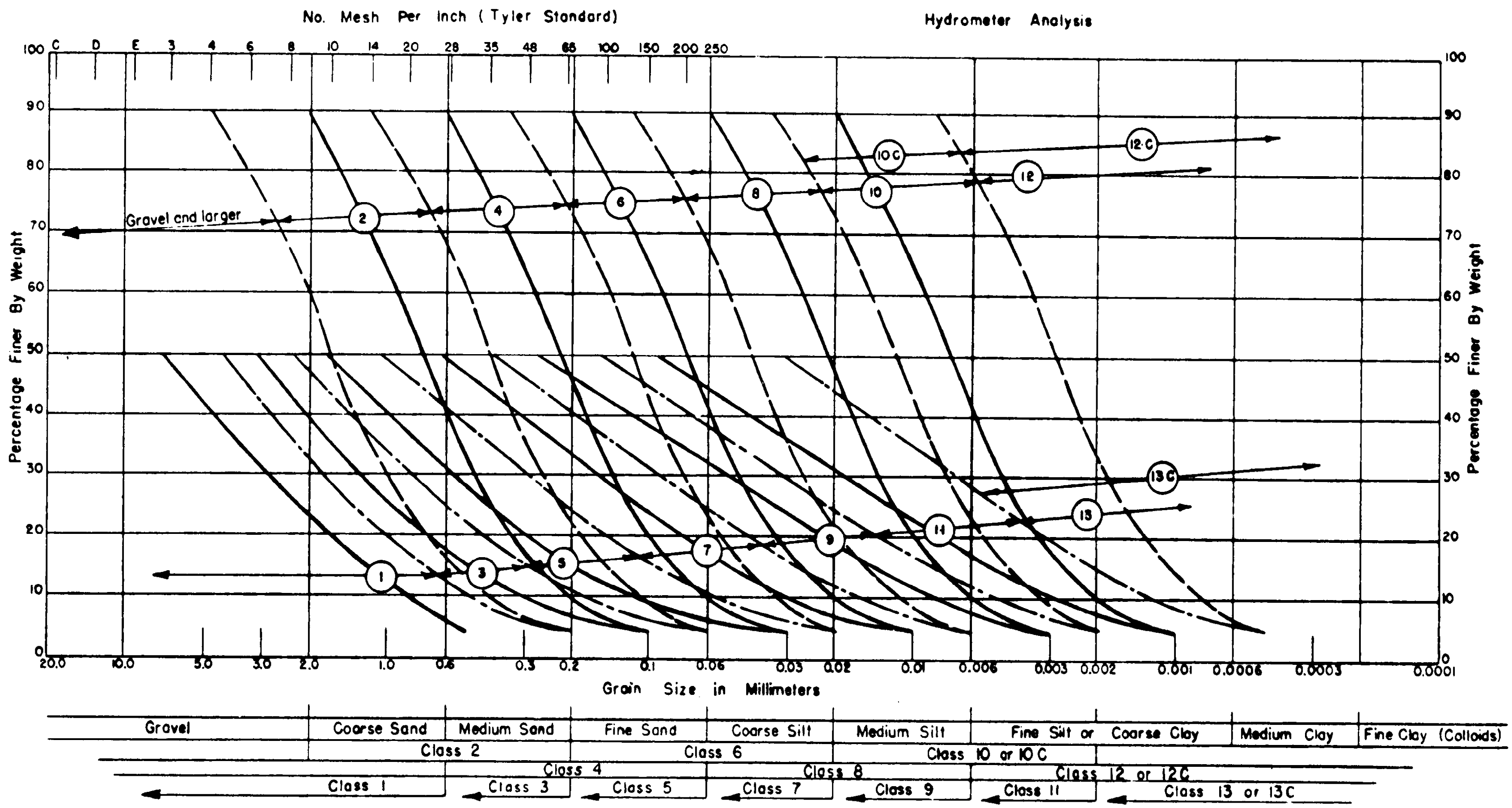
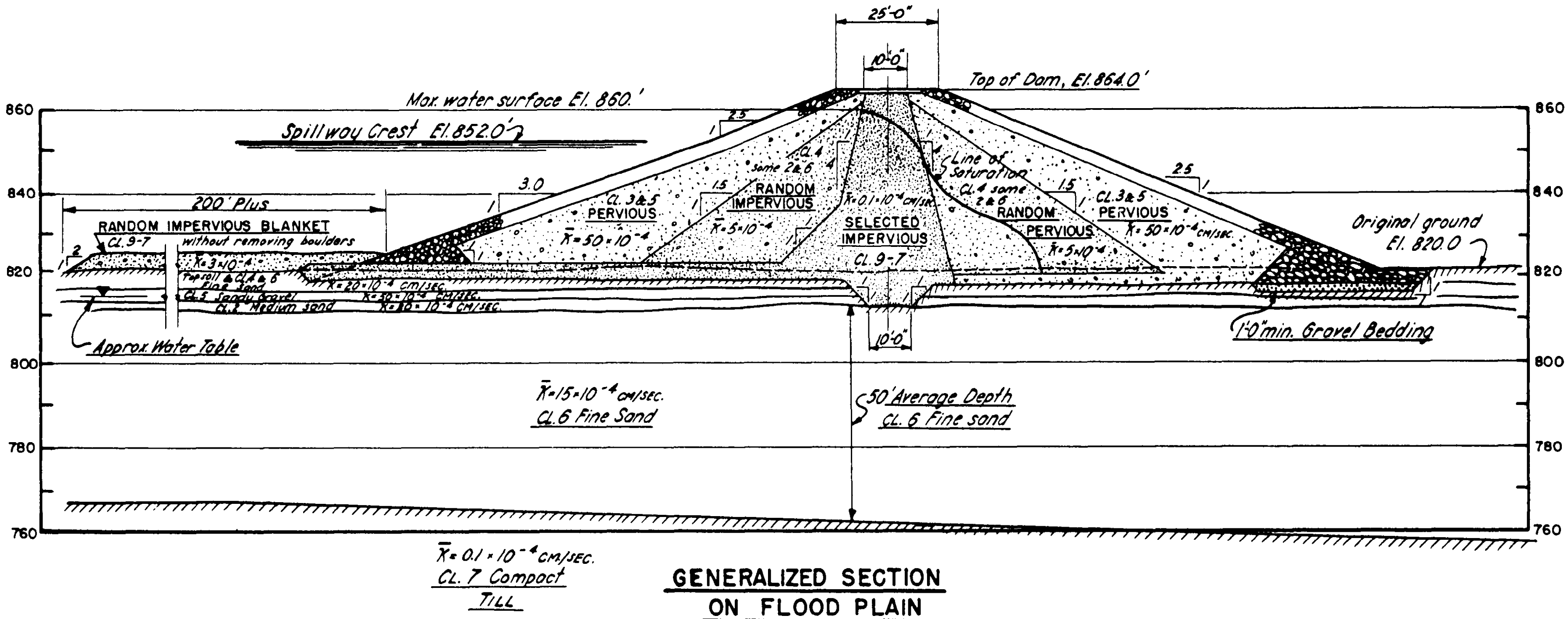


DIAGRAM SHOWING LIMITS OF SOIL CLASSES

PLATE NO.-31



GENERALIZED SECTION ON FLOOD PLAIN

COEFFICIENTS OF PERMEABILITY ARE LISTED AS $\bar{K}$ WHERE $\bar{K} = \sqrt{K_{MAX.} \times K_{MIN.}}$

FOR ASSUMPTIONS $K_{MAX.} = 4\ K_{MIN.}$

$K_{MAX.} = 2\bar{K}$

$K_{MIN.} = \frac{1}{2}\bar{K}$

BIRCH HILL DAM **GENERALIZED SECTION**	
FLOOD CONTROL ENG. DIV., SOILS LAB.	
U.S. ENGINEER OFFICE, PROV., R. I.	
DRAWN BY: P.O.M.	SCALE: 1" = 20'-0"
ANALYSIS BY: K.S.L.	JAN. 2, 1940

S.L. No. BHM - Eld.

PLATE NO. 32

WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY

SHEAR TEST

SUPPLEMENTARY DATA

Rate of strain 0.06+ in/min.

Consolidation Full

Shear Plane Dry

Remarks

None

Class 9-7

ϕ=34°30' c= 0.0 tons per sq. ft.

SITE BIRCH HILL DAM

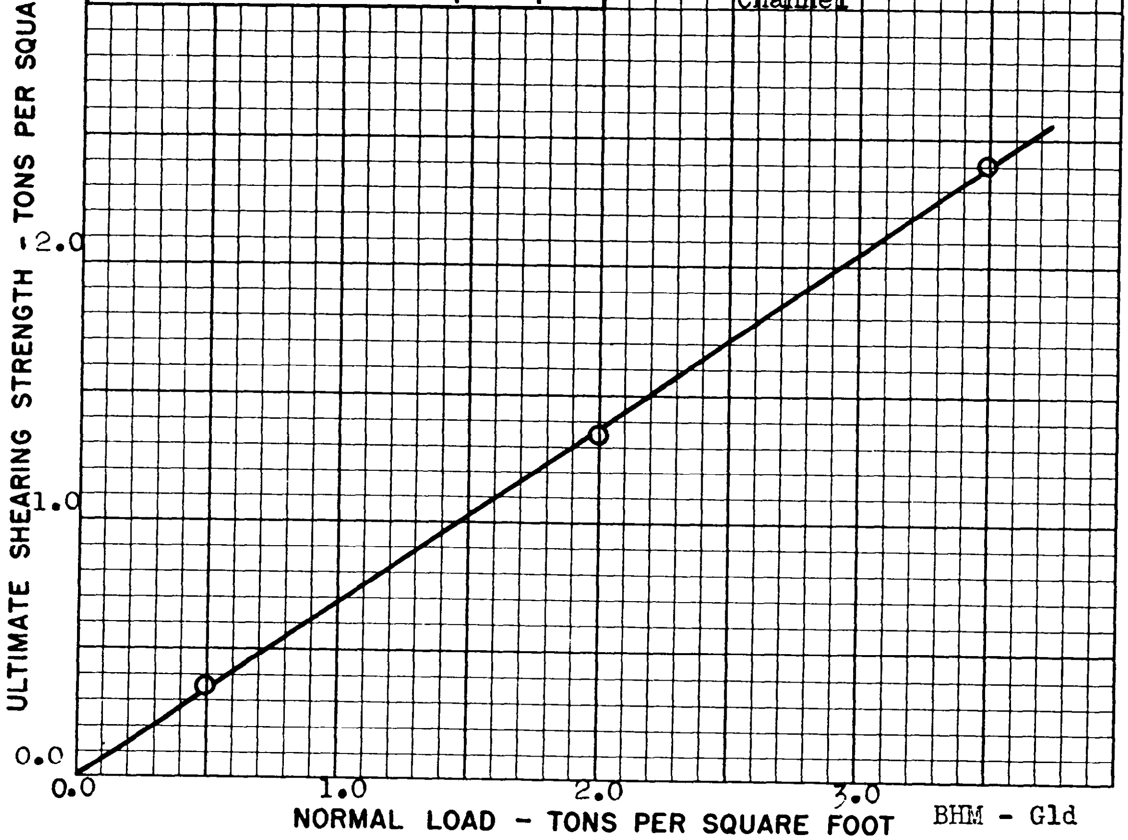
HOLE NO. COMPOSITE

SAMPLE NO. LB1

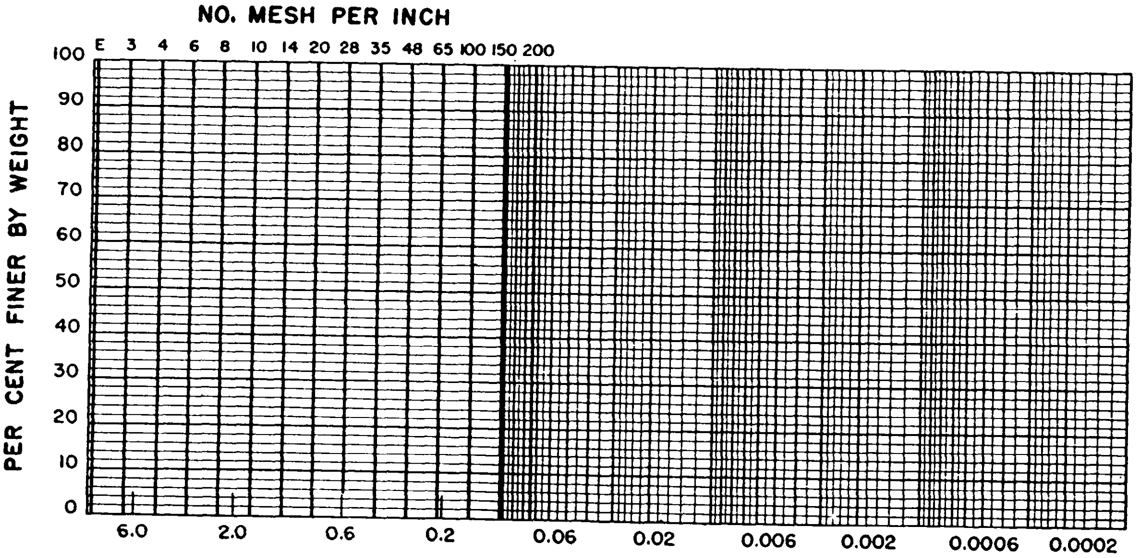
DEPTH

IMPERVIOUS MATERIAL

Excavation for Discharge Channel



BHM - Gld



U. S. GOVERNMENT PRINTING OFFICE 147703

PLATE NO. 33

S.L. FORM NO. 66

FLOOD CONTROL ENGINEERING DIVISION - SOILS LABORATORY

PROVIDENCE, R.I.

SHEAR TEST

SUPPLEMENTARY DATA

Rate of strain 0.06+ in./min.

Consolidation Full

Shear Plane Dry

Remarks None

Class 7-5

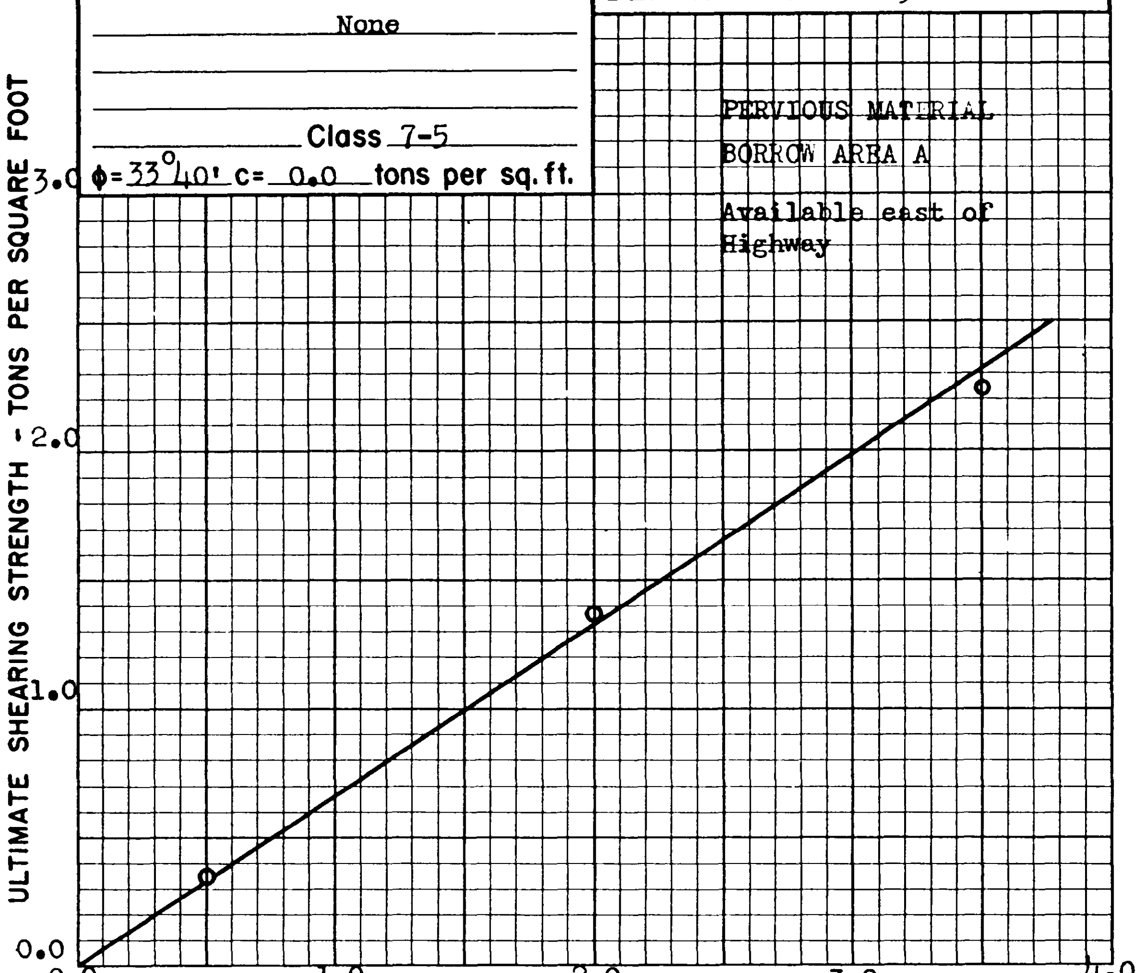
ϕ = 33°40' c = 0.0 tons per sq. ft.

SITE BIRCH HILL DAM

HOLE NO. BT-197

SAMPLE NO. B5 Comp.

DEPTH 0.6' - 8.5'



BHM-G2d

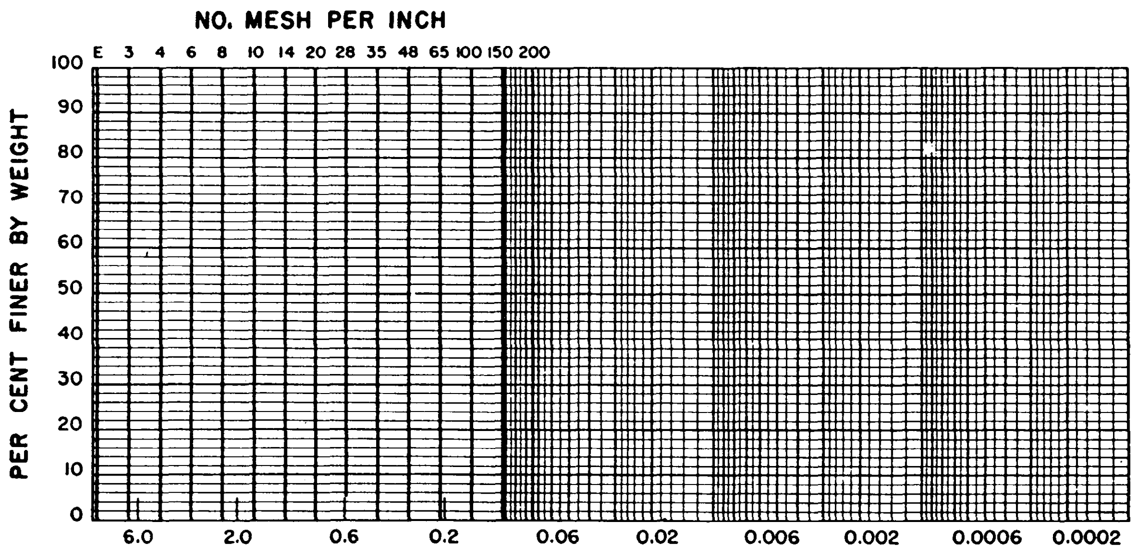


PLATE NO. 34

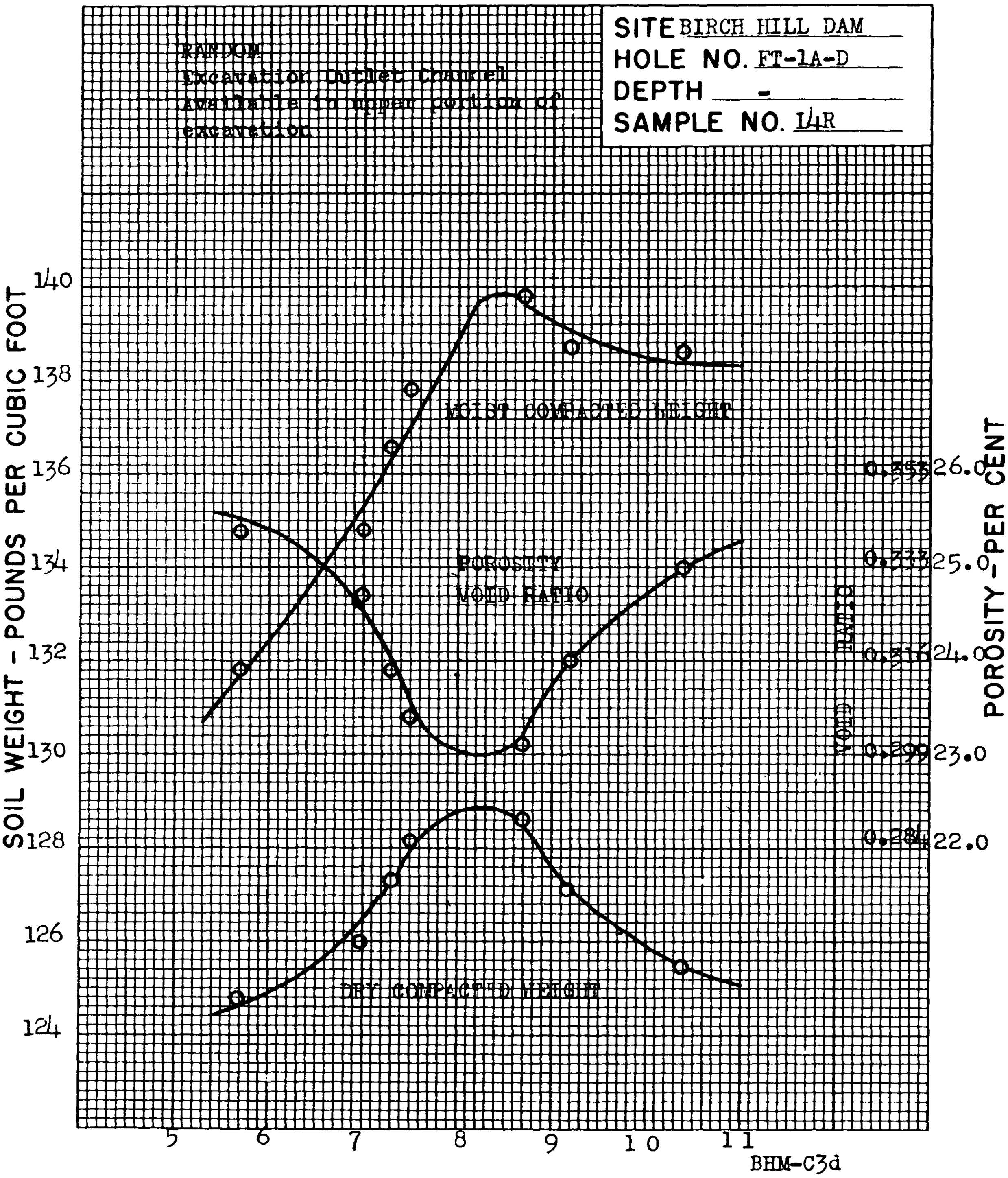
U. S. GOVERNMENT PRINTING OFFICE 147703

S.L. FORM NO. 66

WAR DEPARTMENT CORPS OF ENGINEERS, U. S. ARMY

COMPACTION CHARACTERISTICS

SITE BIRCH HILL DAM
HOLE NO. FT-1A-D
DEPTH -
SAMPLE NO. 14R



WATER CONTENT - PER CENT

Class 9-7

MATERIAL SCREENED OUT

Minimum Size, mm. 6.68
Per Cent by weight 26.0

No. Blows/Layer 25
Area of Tamper, sq. in. 3.14
Weight of Tamper, lbs. 5.5
Fall of Tamper, in. 12.0

n at w opt. = 23.0 %

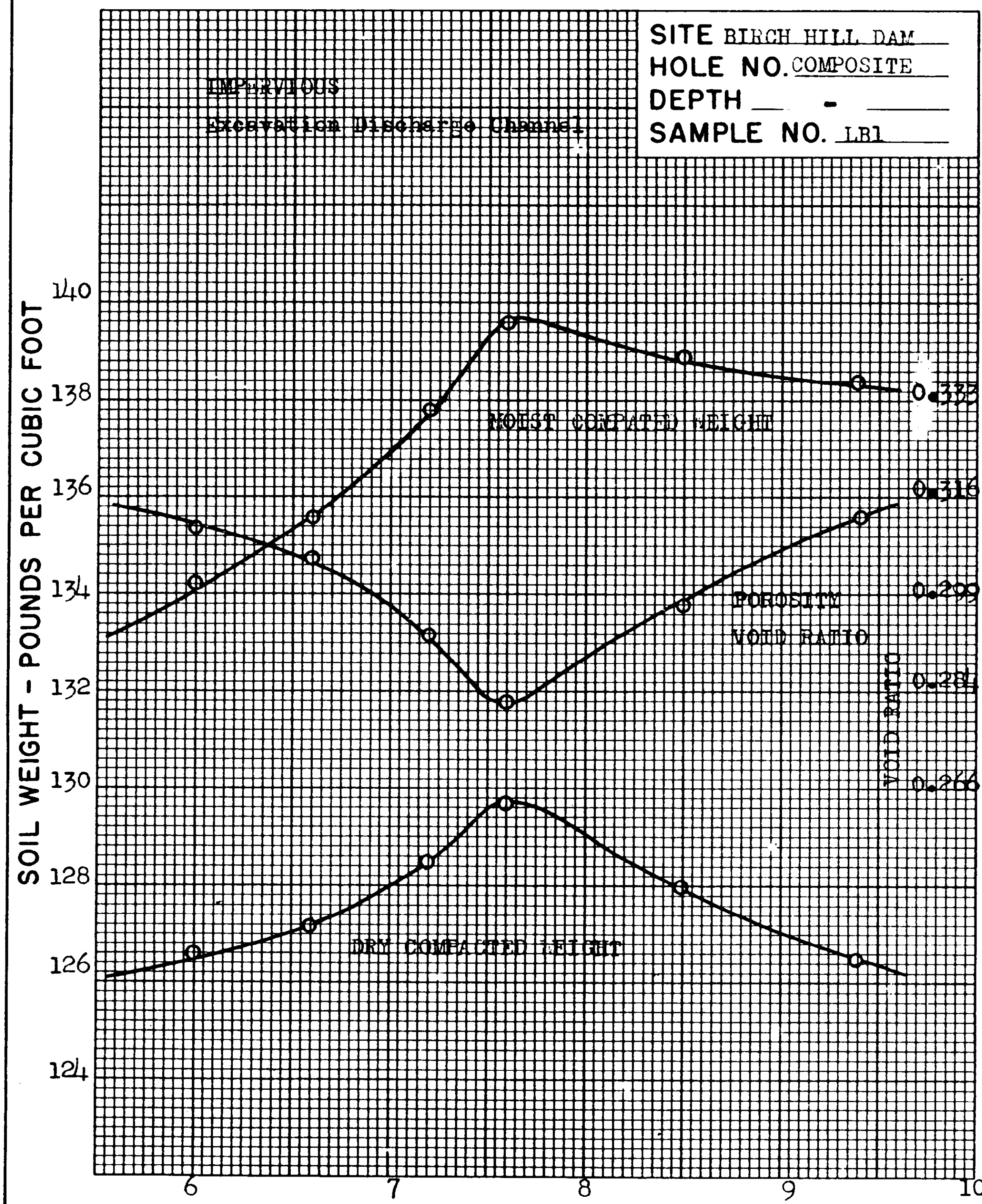
PLATE NO. 35

SOILS LABORATORY PROVIDENCE, R. I.

WAR DEPARTMENT CORPS OF ENGINEERS, U. S. ARMY

COMPACTION CHARACTERISTICS

SITE BIRCH HILL DAM
HOLE NO. COMPOSITE
DEPTH -
SAMPLE NO. LB1



WATER CONTENT - PER CENT BHM-Cld

Class 9-7

No. Blows/Layer 25
Area of Tamper, sq. in. 3.14
Weight of Tamper, lbs. 5.5
Fall of Tamper, in. 12.0

MATERIAL SCREENED OUT
Minimum Size, mm. 6.68
Per Cent by weight 14.6

n at w opt. = 21.9%

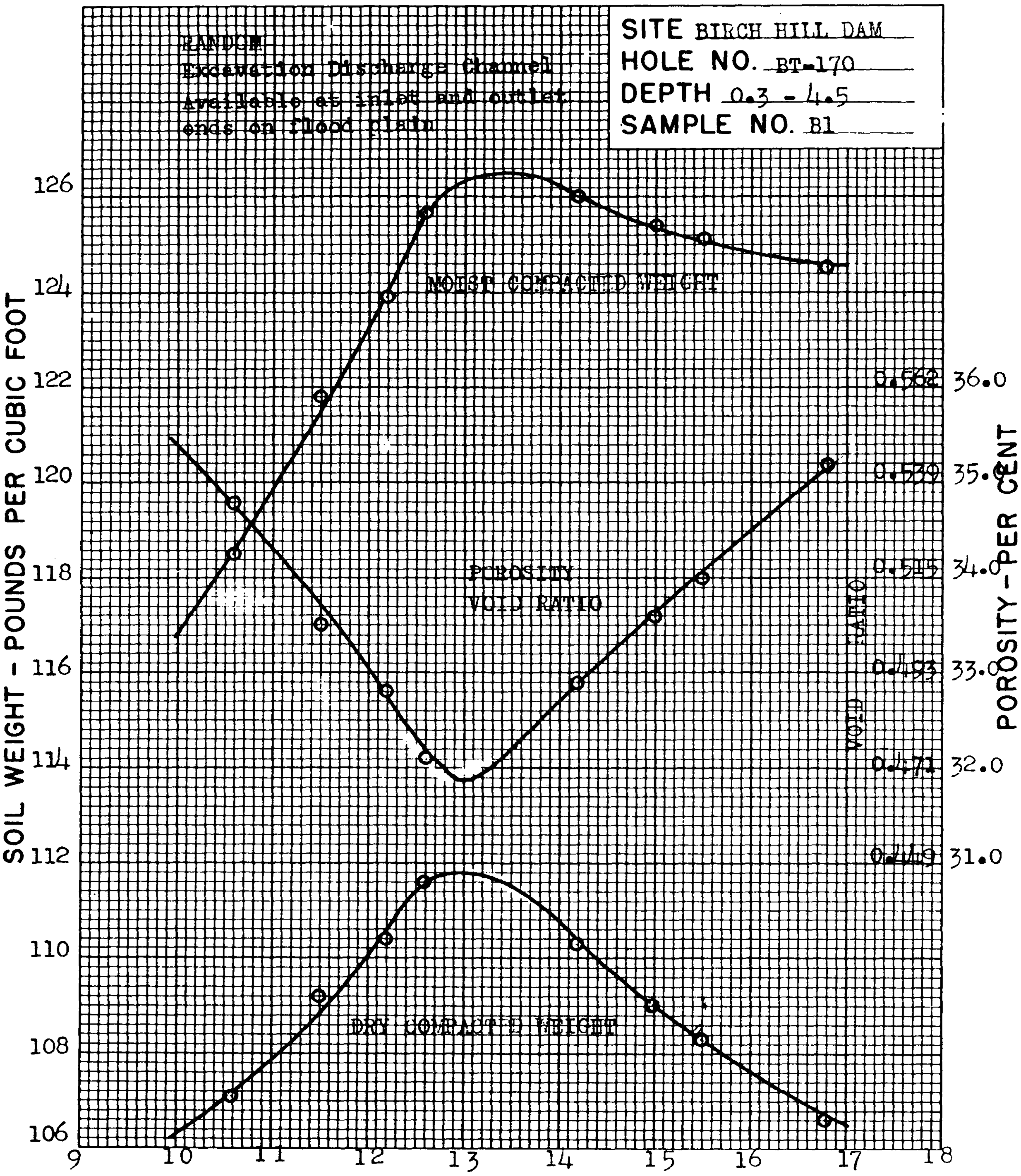
PLATE NO. 36

SOILS LABORATORY PROVIDENCE, R. I.

WAR DEPARTMENT CORPS OF ENGINEERS, U. S. ARMY

COMPACTION CHARACTERISTICS

SITE BIRCH HILL DAM
HOLE NO. BT-170
DEPTH 0.3 - 4.5
SAMPLE NO. B1



WATER CONTENT - PER CENT BHM-C2d

Class 4

MATERIAL SCREENED OUT

Minimum Size, mm. -
Per Cent by weight -

No. Blows/Layer 25
Area of Tamper, sq. in. 3.14
Weight of Tamper, lbs. 5.5
Fall of Tamper, in. 12.0

n at w opt. = 31.8%

PLATE NO. 37

SOILS LABORATORY PROVIDENCE, R. I.

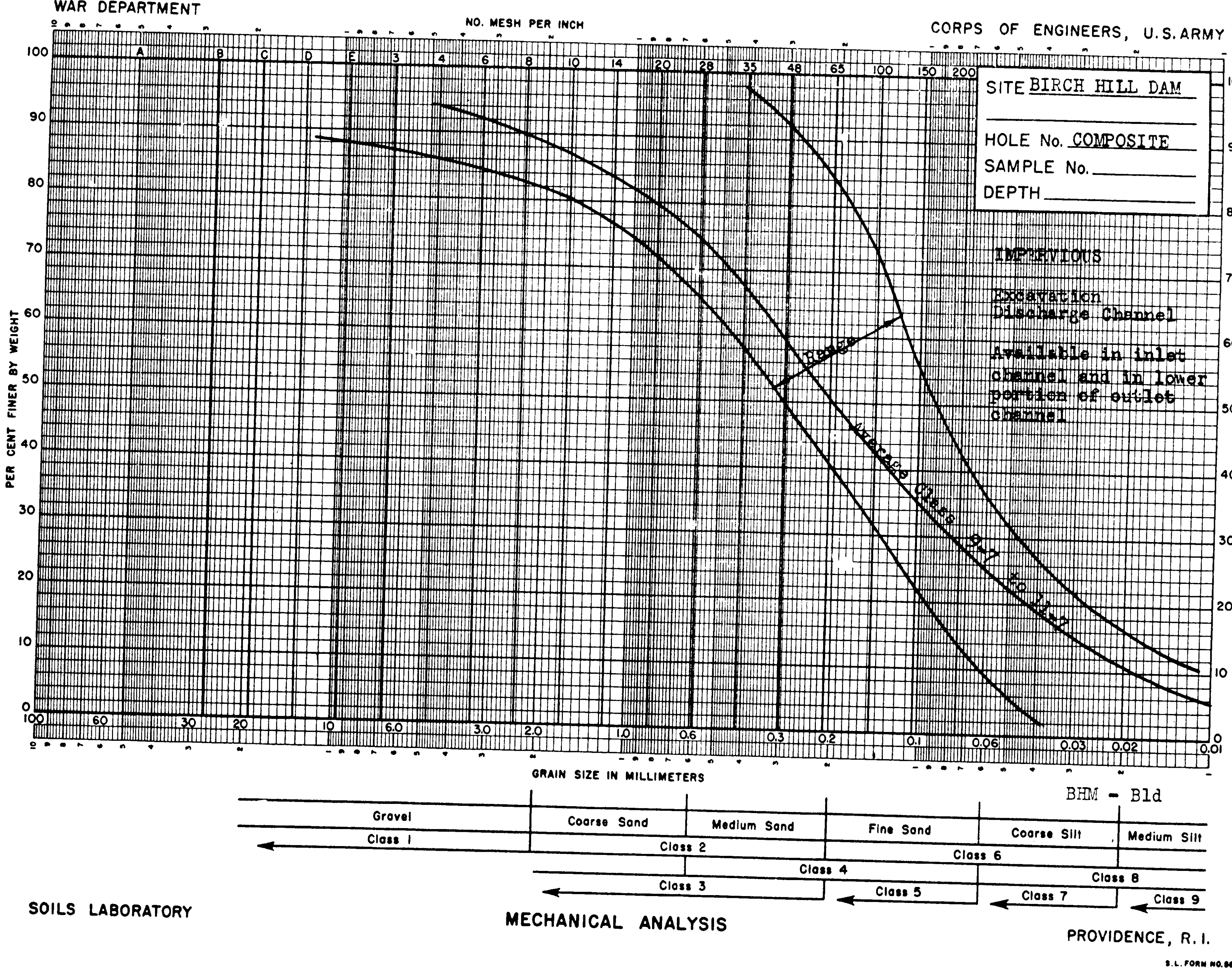
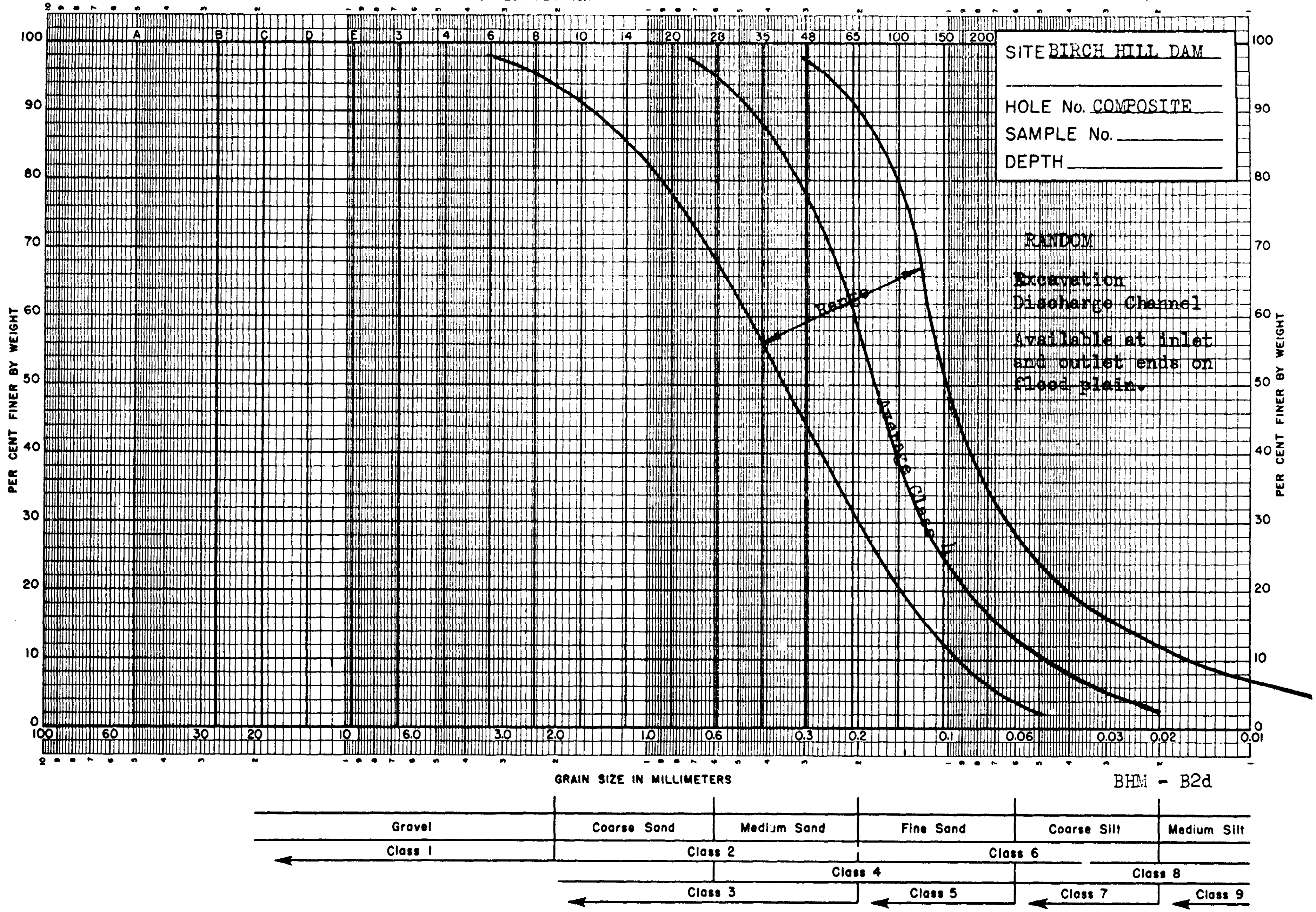

WAR DEPARTMENT
CORPS OF ENGINEERS, U.S. ARMY
NO. MESH PER INCH
GRAIN SIZE IN MILLIMETERS
PER CENT FINER BY WEIGHT
SITE BIRCH HILL DAM
HOLE No. COMPOSITE
SAMPLE No.
DEPTH
IMPERVIOUS
Excavation
Discharge Channel
Available in inlet channel and in lower portion of outlet channel
Range
Average Class 9-7 to 11-7
BHM - Bld
Gravel
Coarse Sand
Medium Sand
Fine Sand
Coarse Silt
Medium Silt
Class 1
Class 2
Class 3
Class 4
Class 5
Class 6
Class 7
Class 8
Class 9
SOILS LABORATORY
MECHANICAL ANALYSIS
PROVIDENCE, R.I.
S.L. FORM NO. 56

PLATE NO. 38

WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY



SOILS LABORATORY

MECHANICAL ANALYSIS

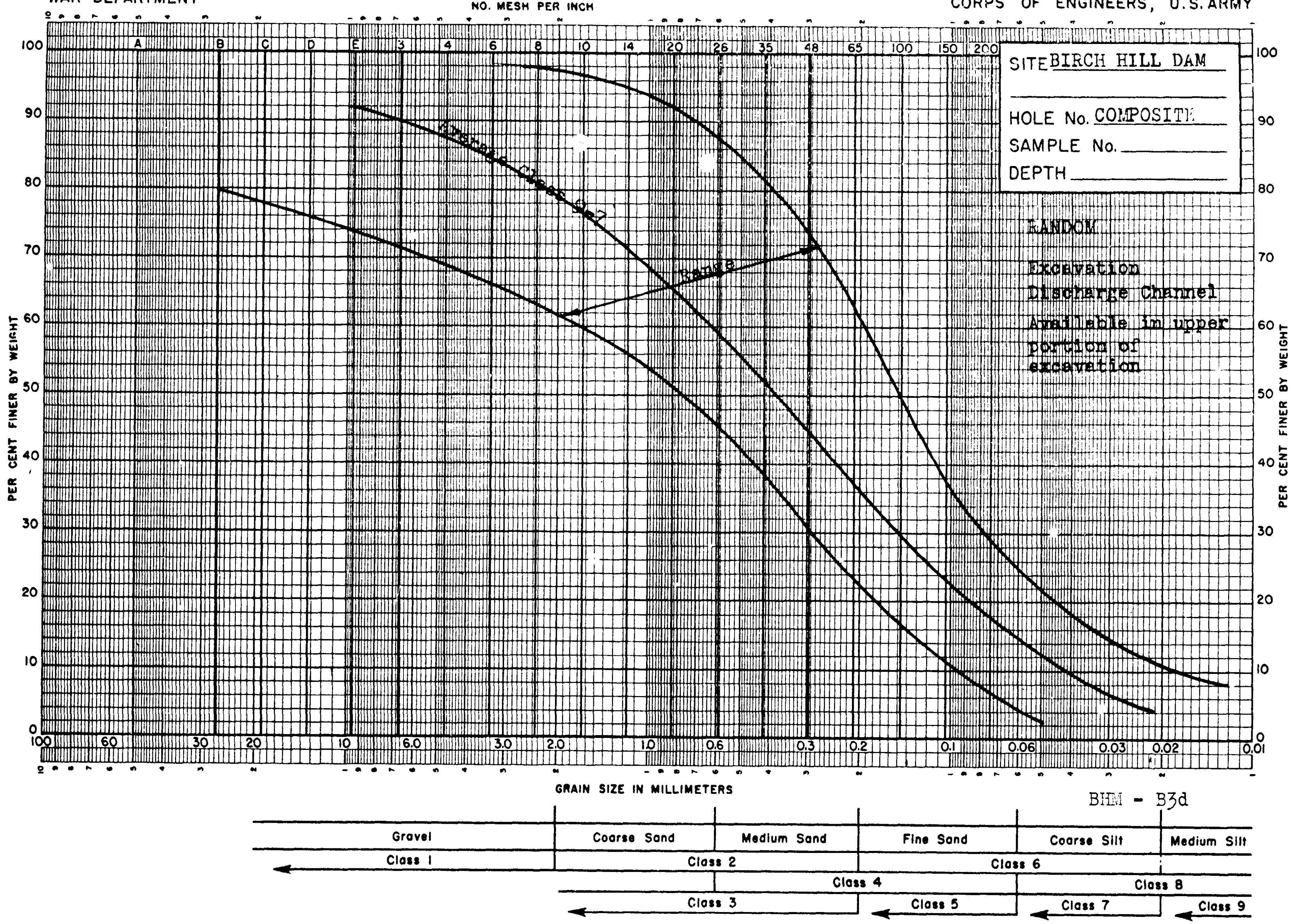
PROVIDENCE, R.I.

S.L. FORM NO. 86

PLATE NO. 39

WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY



SOILS LABORATORY

MECHANICAL ANALYSIS

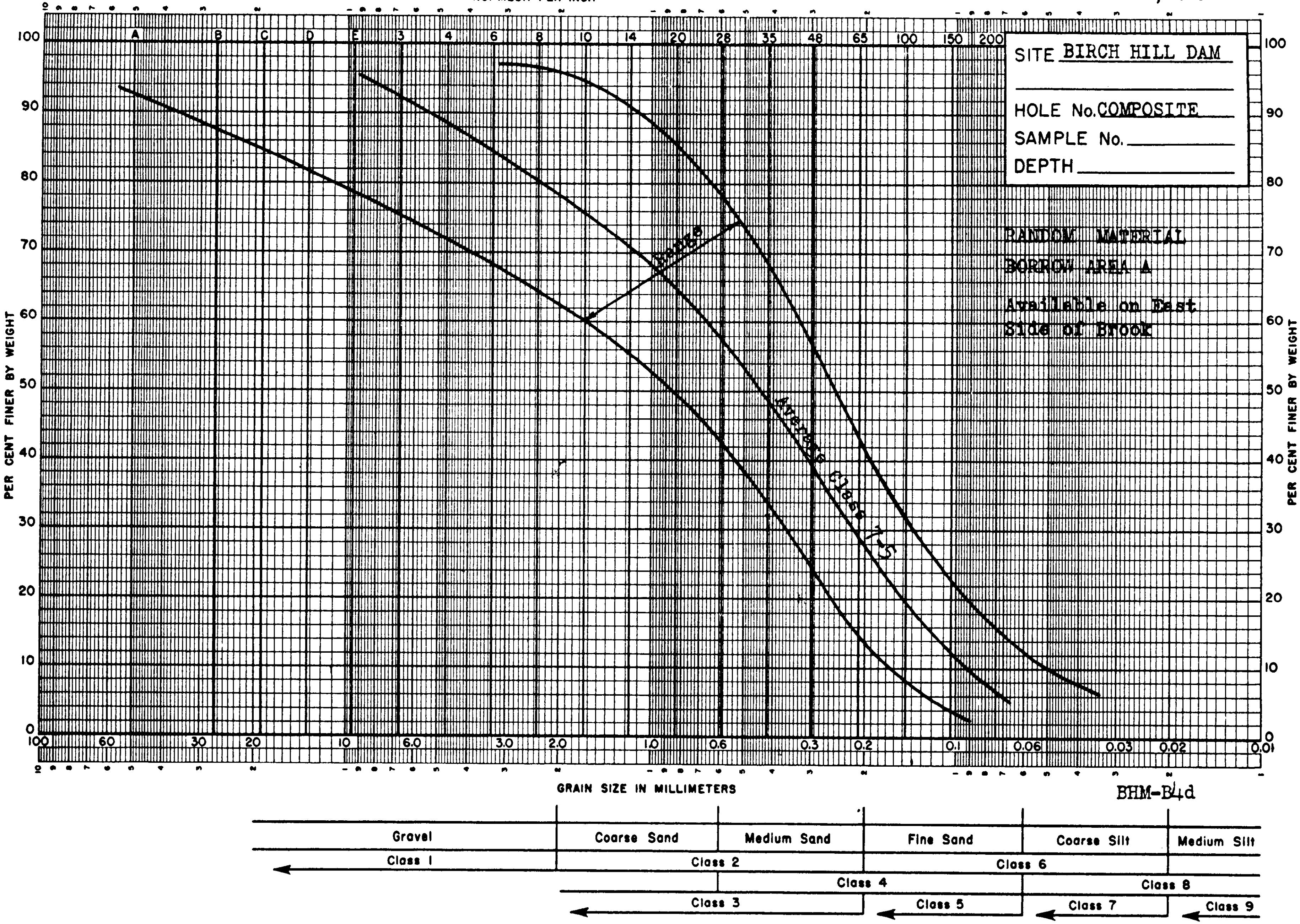
PROVIDENCE, R.I.

S.L. FORM NO. 56

PLATE NO. 40

WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY



SOILS LABORATORY

MECHANICAL ANALYSIS

PROVIDENCE, R.I.

S.L. FORM NO.96

PLATE NO. 41

WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY

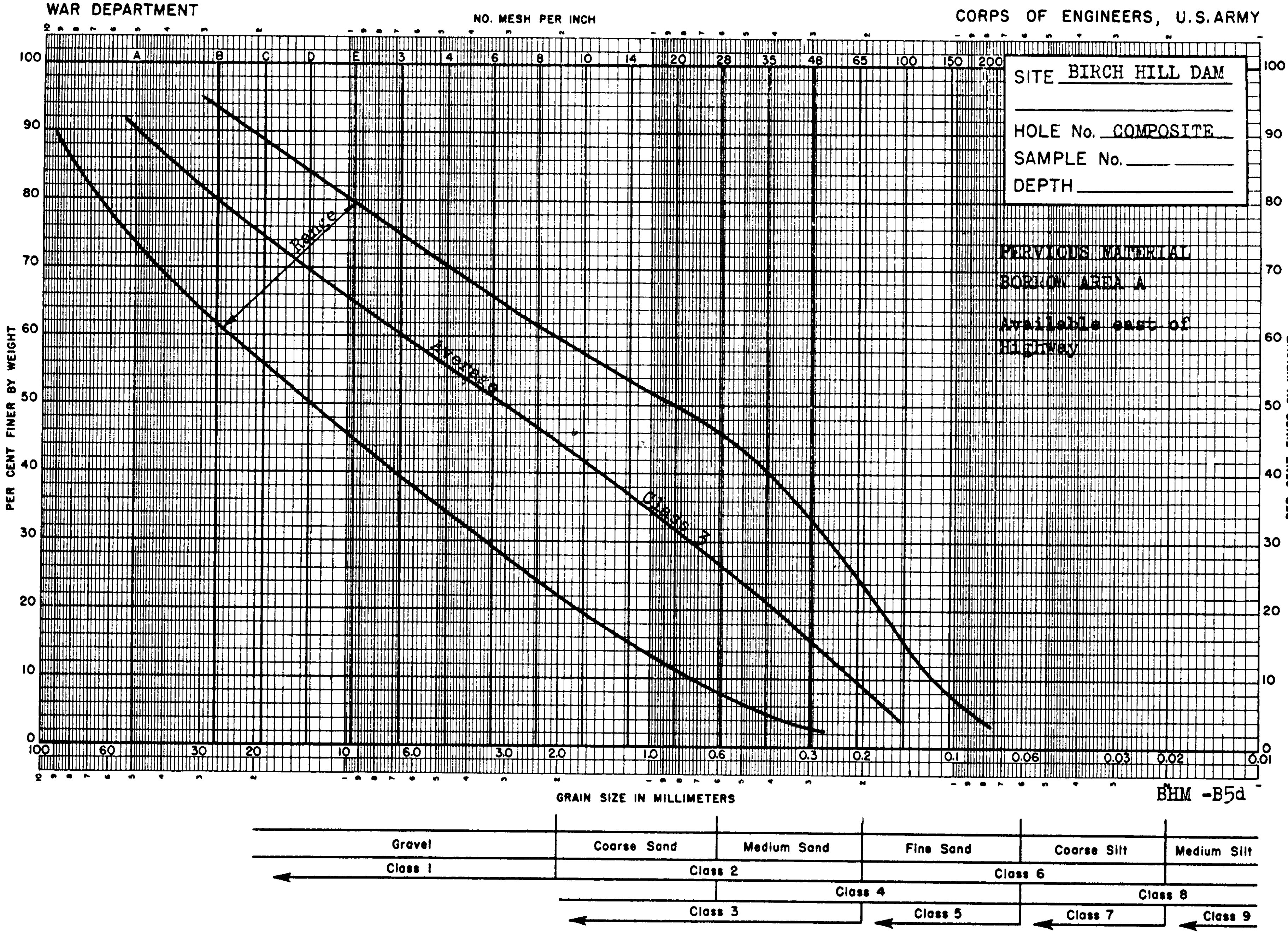


PLATE NO. 42

SOILS LABORATORY

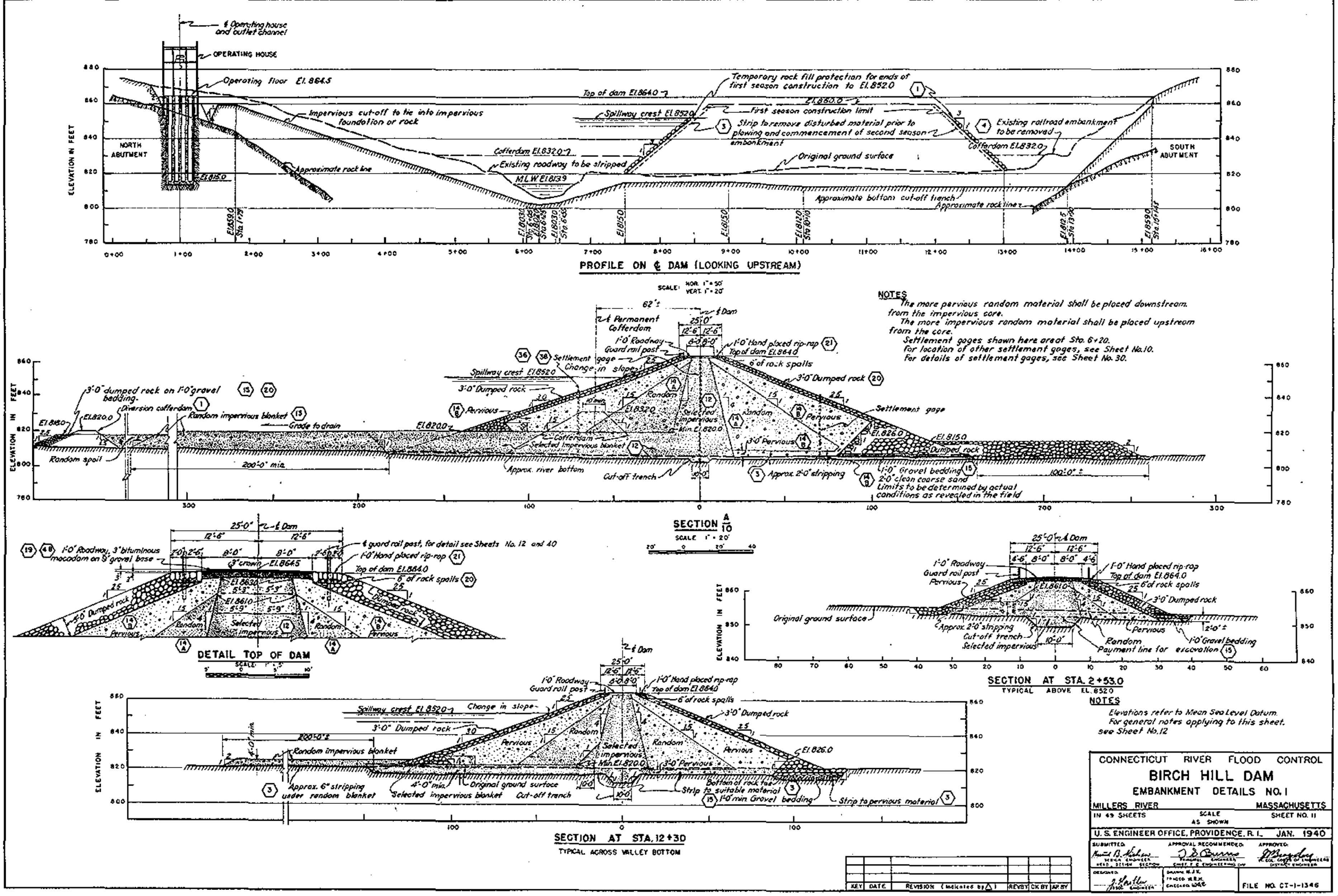
MECHANICAL ANALYSIS

PROVIDENCE, R. I.

S.L. FORM NO. 86

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



NOTES

Elevations refer to Mean Sea Level Datum.
For general notes applying to this sheet, see Sheet No. 12

CONNECTICUT RIVER FLOOD CONTROL

BIRCH HILL DAM

EMBANKMENT DETAILS NO. 1

MILLERS RIVER — MASSACHUSETTS

IN 49 SHEETS — SCALE AS SHOWN — SHEET NO. 11

U. S. ENGINEER OFFICE, PROVIDENCE, R. I. — JAN. 1940

SUBMITTED — APPROVAL RECOMMENDED — APPROVED

FILE NO. CT-1-1346

PLATE NO. 43

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

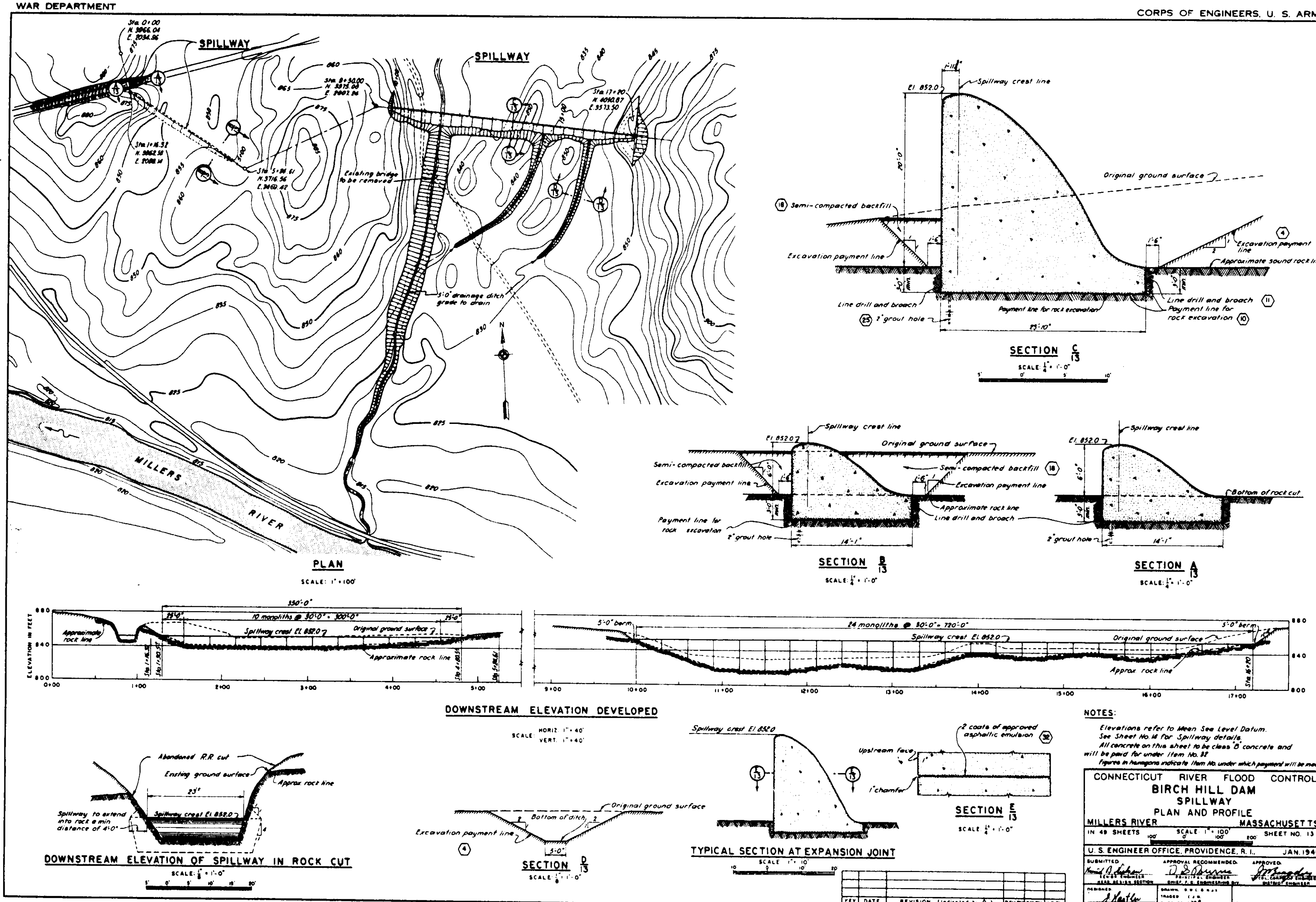
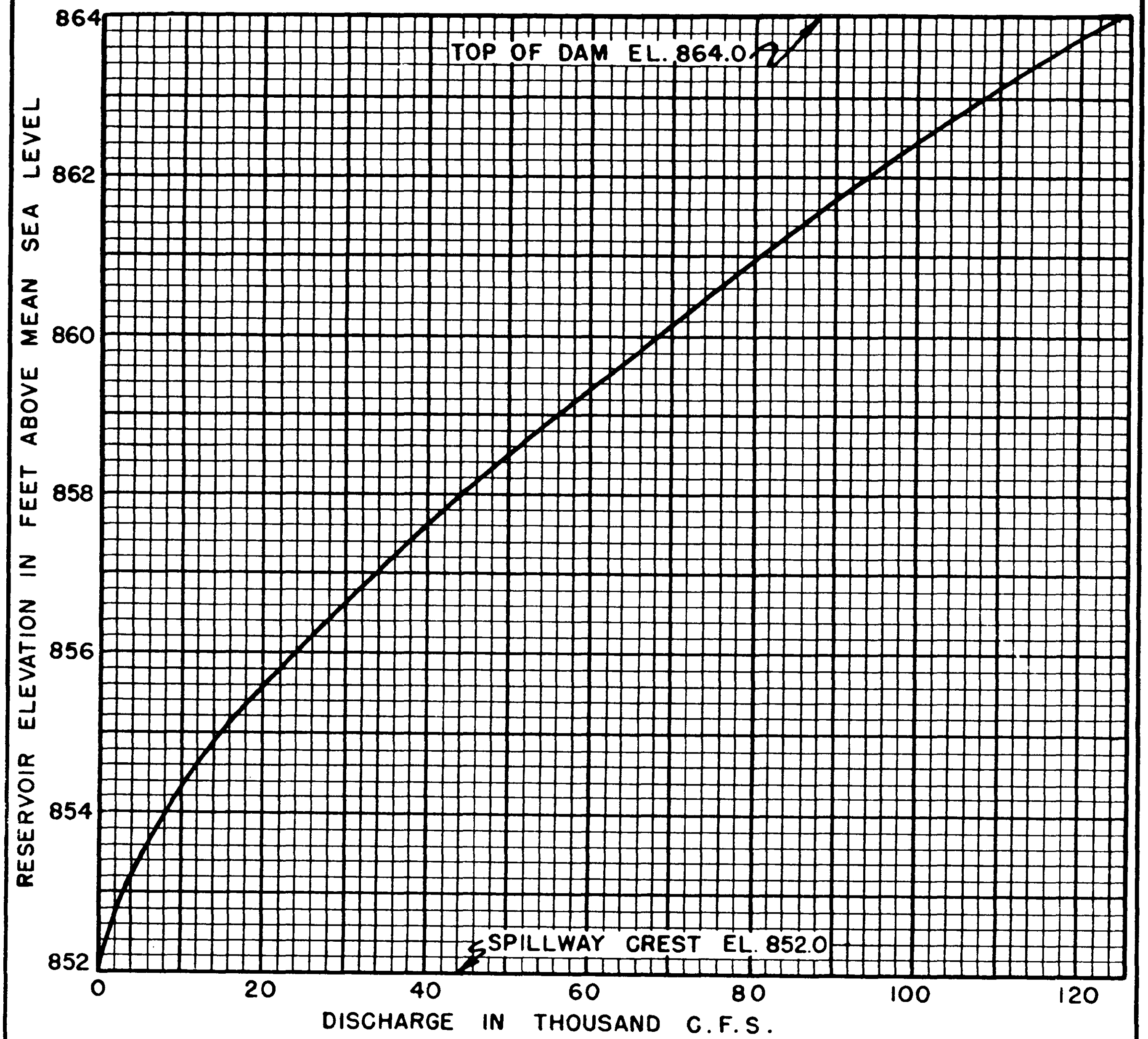


PLATE NO. 44



CONNECTICUT RIVER FLOOD CONTROL

BIRCH HILL DAM

SPILLWAY RATING CURVE

U.S ENGINEER OFFICE, PROVIDENCE, R. I.

PLATE NO. 45

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

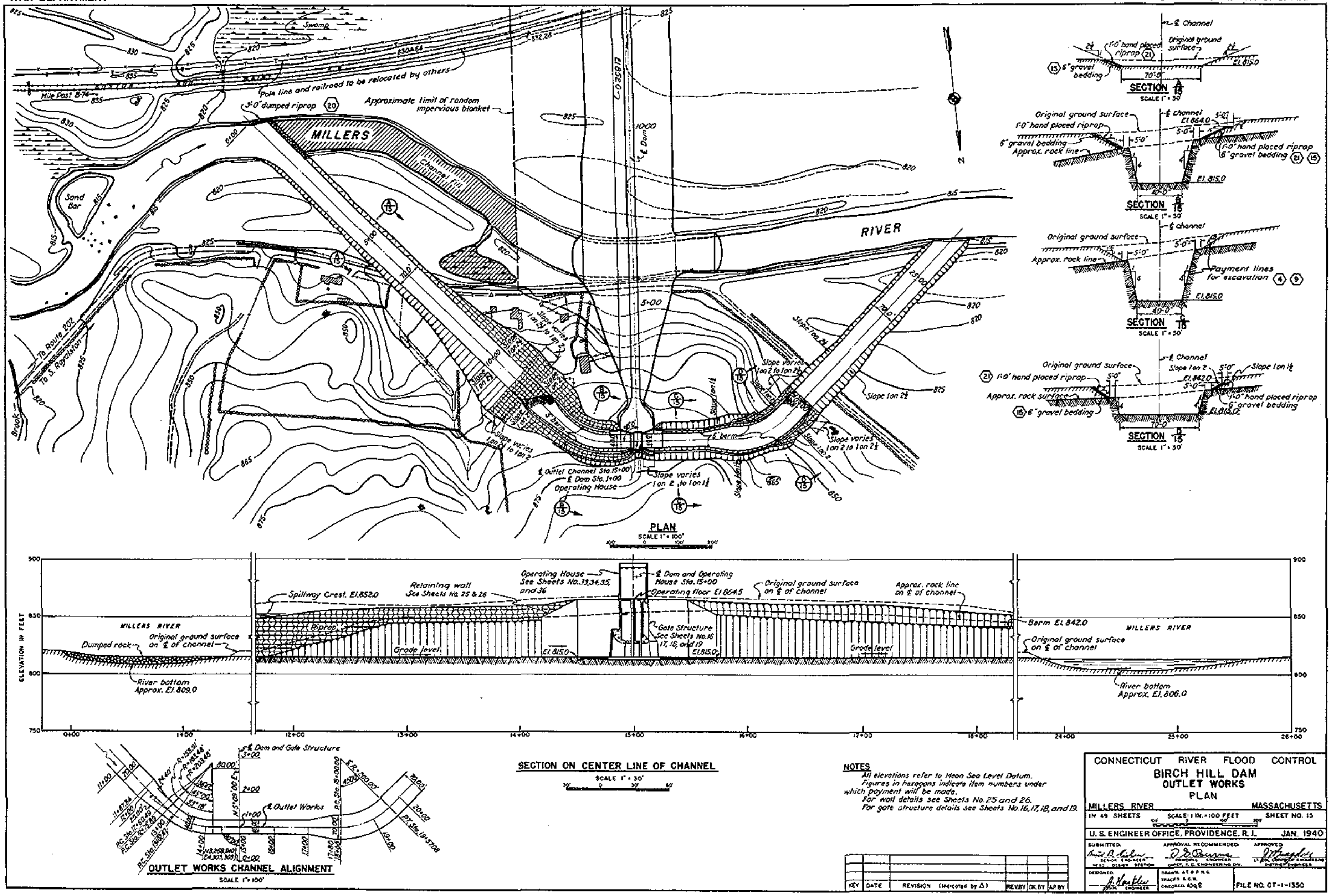
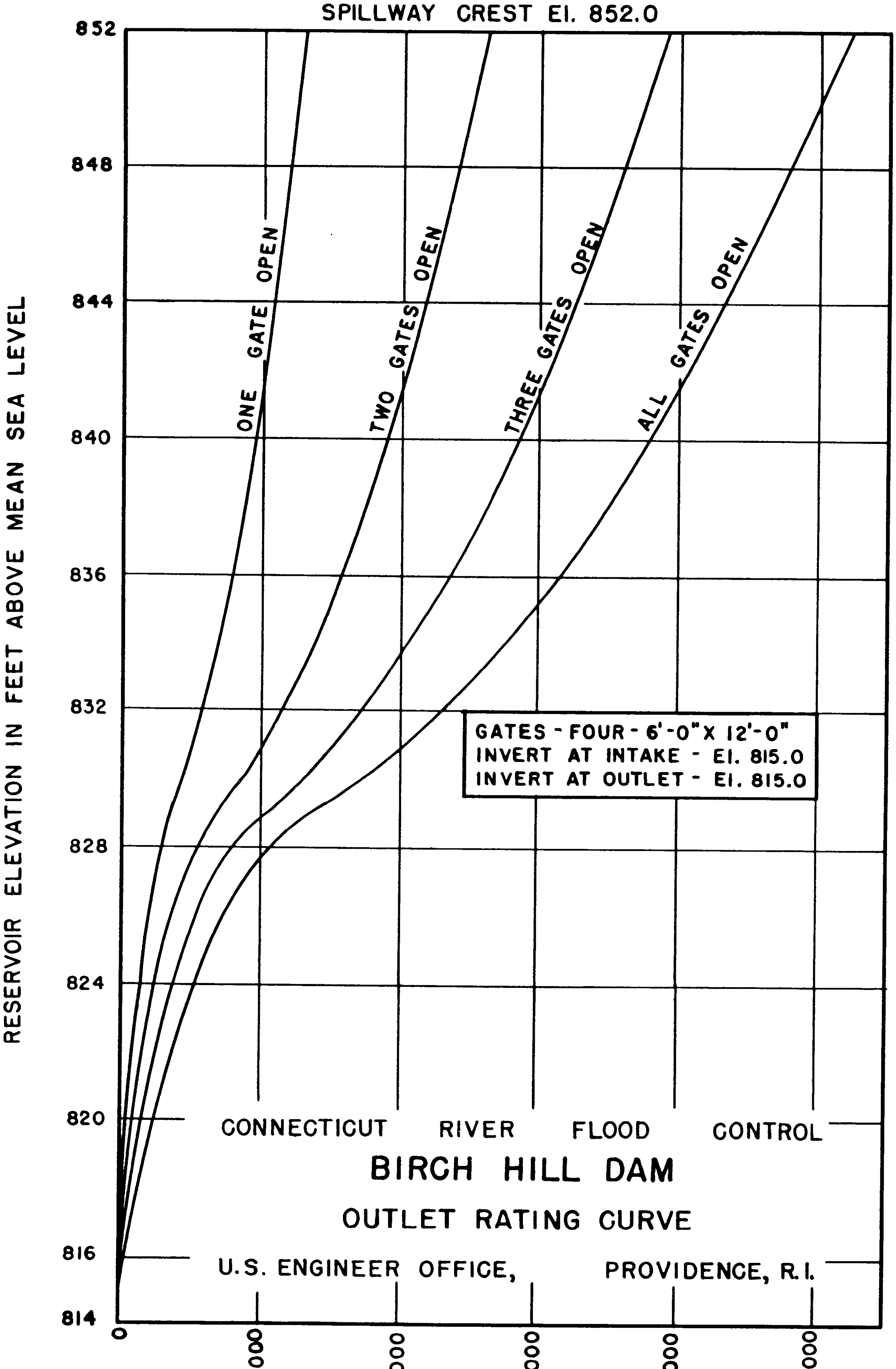


PLATE NO. 46

G.49

(15)



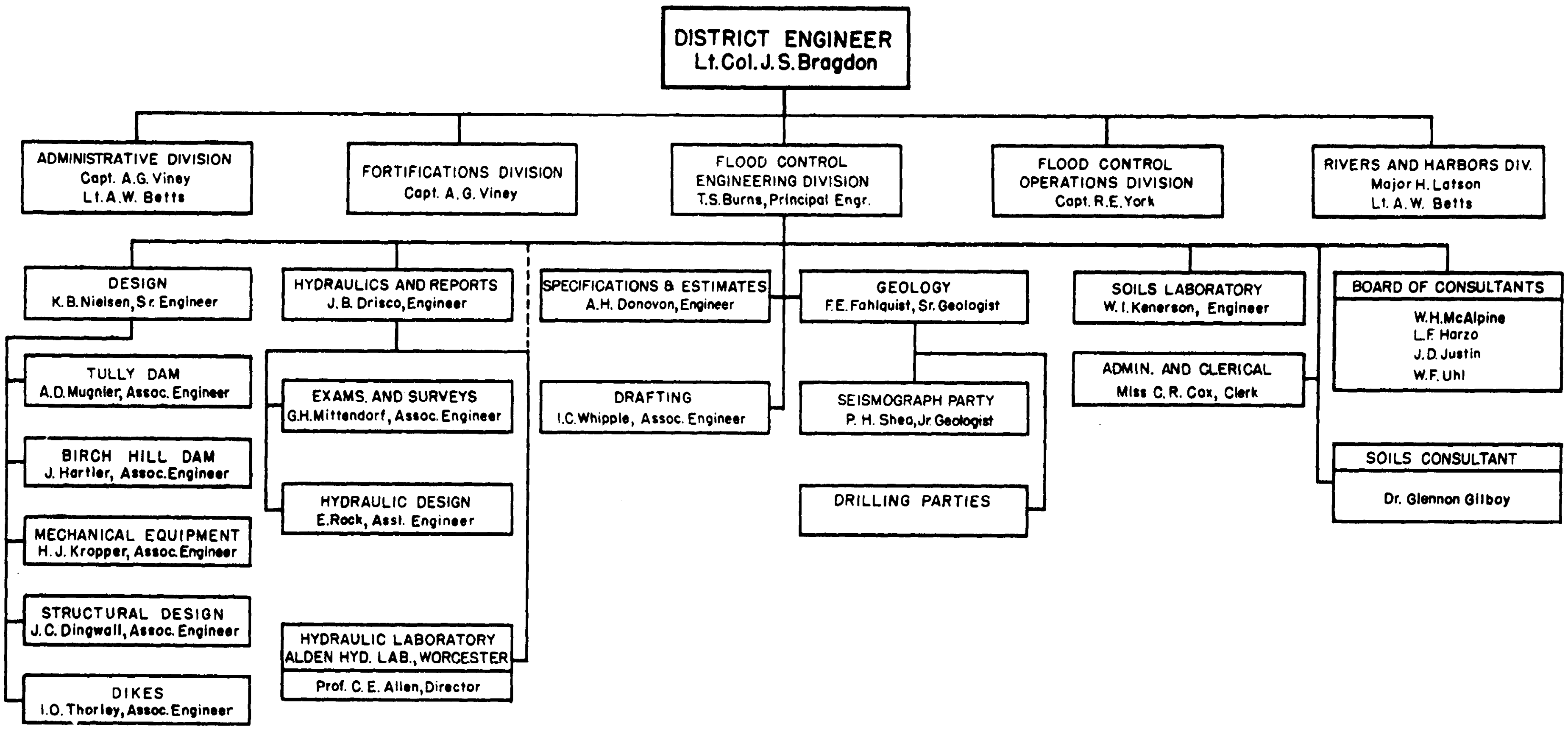
CONNECTICUT RIVER FLOOD CONTROL

BIRCH HILL DAM

OUTLET RATING CURVE

U.S. ENGINEER OFFICE, PROVIDENCE, R.I.

PLATE NO. 47



ORGANIZATION CHART

FLOOD CONTROL ENGINEERING DIVISION

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.
APRIL 1940

PLATE NO. 48

CONNECTICUT RIVER FLOOD CONTROL

BIRCH HILL DAM

MILLERS RIVER
MASSACHUSETTS

ANALYSIS OF DESIGN

APPENDIX A

1940

CORPS OF ENGINEERS, U. S. ARMY

U. S. ENGINEER OFFICE PROVIDENCE, R. I.

BIRCH HILL DAM

ANALYSIS OF DESIGN

STRUCTURAL DESIGN COMPUTATIONS

APPENDIX A

TABLE OF CONTENTS

INTRODUCTION - STRUCTURAL DESIGN STANDARDS

1. Structures investigated. - Features of the dam for which structural analysis is included are the outlet works, including gate structure, operating house, and retaining walls, the spillway weir, and the access road bridge.

2. Design standards. - Standard practice in structural design has been followed throughout. In general, concrete design is based on the "Joint Code of the American Concrete Institute and the Reinforcing Steel Institute for the Design of Concrete and Reinforced Concrete", issued in 1928, and the "Progress Report" of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, issued in January, 1937. Steel design is based on the "Standard Specifications for Design and Fabrication of Structural Steel", issued by the American Institute of Steel Construction. Allowable stresses are low to insure permanence in accordance with the character of the project.

3. Loadings. - a. Dead loads. - Dead loads include weights of structures, and materials exclusive of gates and accessories. Unit weights are as follows:

Material	Unit weight pounds per cu. ft.
Concrete	150
Brick	130
Water	62.5
Dry earth	100
Saturated earth	125

b. Live loads. - H-15 loading was used for the design of the access road bridge.

The operating floor of the gate structure was designed for live load produced in placing the heaviest item of equipment on the floor, and amounted to 250 pounds per square foot. Floor beams were designed for a loading of 80 percent of 250 pounds per square foot or 200 pounds per square foot. Gate pull was 47,000 pounds.

Snow load on the roof of the operating house was assumed at 40 pounds per square foot.

Wind loading on exposed vertical faces of the operating house was assumed at 30 pounds per square foot.

c. Earth pressures. - Horizontal earth pressures were computed by the use of Rankine's formula, resulting in a lateral loading of 35 pounds per square foot for dry earth and 80 pounds per square foot for saturated earth.

d. Hydrostatic pressure. - The gate structure is designed for stability against hydrostatic head to reservoir surcharge elevation, which proved to be the severest condition of loading. No allowance is made for restraint due to frictional resistance against the rock side walls. The design of walls and piers was made for hydrostatic loading to spillway elevation and checked against surcharge loading with 33-1/3 percent increase in allowable stresses. This increase in allowable stresses is made on account of the infrequency and short duration of such a loading condition. Side walls were designed for full hydrostatic pressure for 10 feet into rock and 50 percent hydrostatic pressure beyond that depth, this being an assumption of the effectiveness of adhesion between rock and concrete.

The spillway is designed for stability against hydrostatic head to surcharge elevation.

In stability computations uplift assumptions were as follows: 100 percent hydrostatic head at upstream face, 100 percent tailwater head at downstream toe, and tailwater head plus 50 percent of the difference between tailwater and headwater at a point 10 feet from the upstream face, varying uniformly between these points.

4. Allowable stresses. - a. General statement. - The specified 28-day compressive strength for Class "A" concrete used in the gate structure and bridge is 3,400 pounds per square inch. For added permanence and safety the allowable design stresses have been reduced, and are as listed below. Structural steel stresses are in accordance with standard design.

b. Reinforced concrete. -

	Lbs. per sq. in.
(1) Flexure. - Extreme fiber stress in compression	1000
(2) Shear. - Beams with no web reinforcement and without special anchorage of longitudinal steel	50
Beams with no web reinforcement but with special reinforcement of longitudinal steel	75
Beams with properly designed web reinforcement but without special anchorage of longitudinal steel	150
Beams with properly designed web reinforcement and with special anchorage of longitudinal steel	225

Lbs. per sq. in.

(3) Bond. - In beams and slabs (Deformed bars used throughout) (Where special anchorage is provided double these values in bond may be used) 125

(4) Bearing. - Where a concrete member has an area at least twice the area in bearing 625

(5) Axial compression. - In columns with lateral ties 563

(6) Allowable unit stresses in reinforcement

	Ordinary structures	Structures much exposed to water
Tension	18,000	16,000
Web reinforcement	16,000	16,000

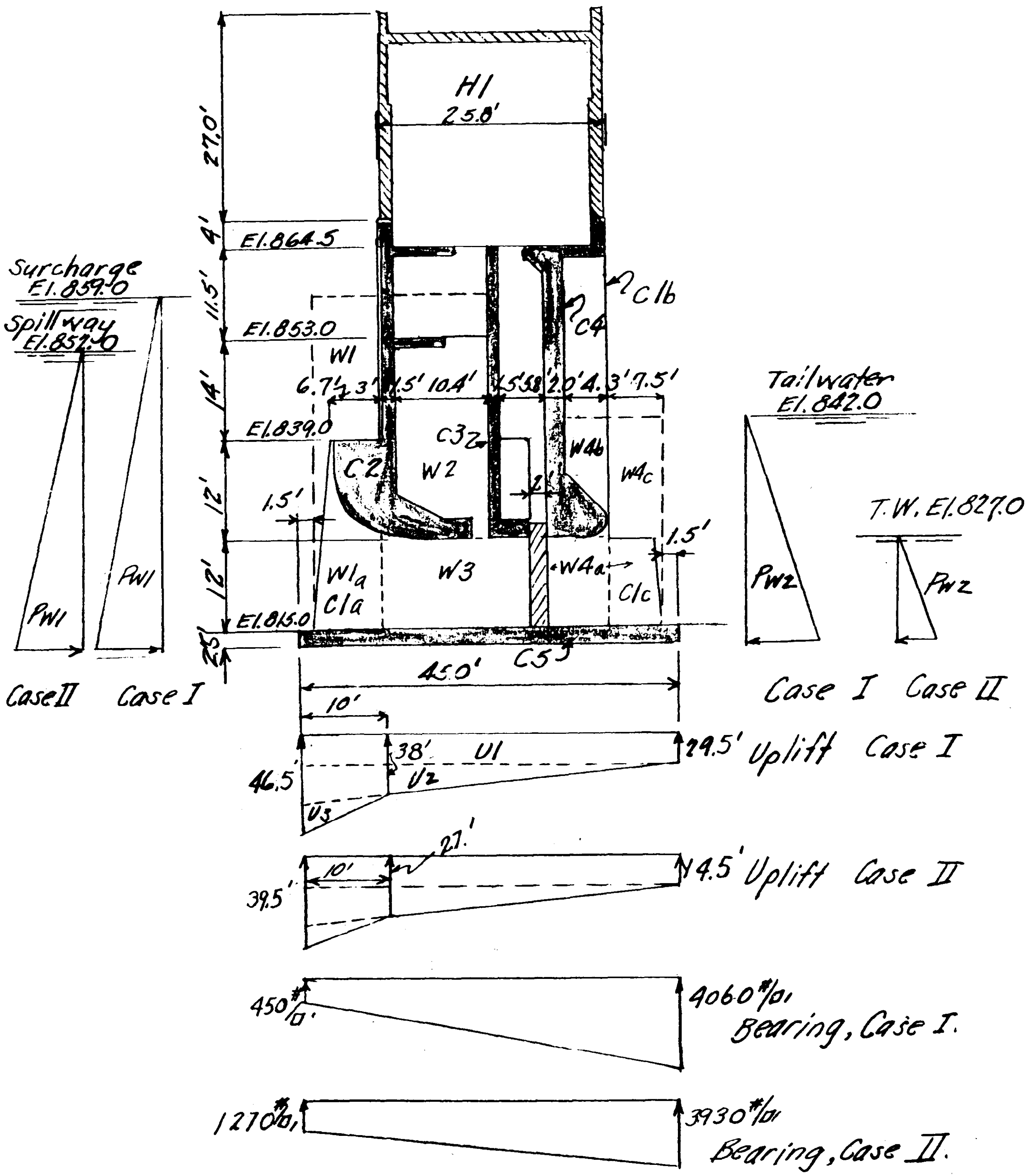
c. Structural steel. - Allowable stresses have been maintained in accordance with the "Standard Specifications for Steel Construction" of the American Institute of Steel Construction.

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page 5

ject BIRCH HILL
nputation Gate tower
mputed by RSM   Checked by RHM   Date 12-1-39



Subject: Birch Hill
Computation: Gate tower
Computed by: RSM Checked by: R.H.M. Date: 12-1-39

STABILITY - CASE I - Water at surcharge elevation
Consider 1 complete pier and intermediate bay as a section = 10' wide
Preliminary studies indicate most severe condition exists with water at surcharge elevation, when stability is being considered

	Force	Am't. (kips) ↓	Am't. (kips) ↑	Am't. →	Am't. ←	Arm	Moments abt. "A" (ft. kips) ↷	Moments abt. "A" (ft. kips) ↶
H1	130×1.5×27×10×2	105.5				21.9		2315
	150×1.0×23.8×10	35.7				21.9		784
C1a	150×7.7×3.0×24	83.1				38.7		3215
C1b	150×12.2×4×38	279.0				28.7		8007
	150×13.6×4×49.5	405.0				15.8		6400
	-(150×5.8×2×25.5)		44.4			18.2	808	
C1c	150×6.9×3×12	37.3				5.9		220
C2	150×1.2×8×10	14.4				29.0		417
	150×1×6×5.5	4.9				30.2		148
	150×2.8×6×8	20.2				29.6		598
	150×1.8×10×4	10.8				33.9		366
	150×1.7×11.5×10	29.3				33.9		993
	150×1.5×14×6	18.9				33.8		640
	150×76□'×7	79.8				37.0		2950
C3	150×1.5×6×37.5	50.6				21.9		1108
	150×4×2×6	7.2				19.1		137
C4	150×4×1.8×10	10.8				9.9		107
	150×1.5×6×6	8.1				13.3		108
	150×2.0×6×37.5	67.5				14.3		966
	150×22□'×6	19.8				11.5		228
C5	150×2.5×10×45	168.8				22.5		3800
W1	62.5×10.4×10×20	130.1				39.9		5190
W1a	62.5×10.2×8.8×18	101.0				40.1		4060
	62.5×4×6×10.4	15.6				27.8		435
W2	62.5×6×29×10.4	113.3				27.8		3160
W3	62.5×6×12×17.5	78.8				26.0		2050
W4a	62.5×6×12×15.3	69.0				7.6		525
W4b	62.5×6×10×4.3	16.1				11.1		180
W4c	62.5×10×15×9.0	84.4				4.5		380
U_1	62.5×10×29.5×45		830			22.5	18680	
U_2	62.5×10×45×11/2		155			30.0	4640	
U_3	62.5×10×6×10/2		19			41.7	793	
P_{w1}	$62.5 \times 46.5^2 \times 10/2$			676		15.5	10480	
P_{w2}	$62.5 \times 29.5^2 \times 10/2$				272	9.8		2670
		2065.2	1048.4				35401	52157
	ΣV =	1016.8^k ↓				ΣM =		16756

ıbject BIRCH HILL
ɱputation Gate tower
ɔmputed by RSM Checked by R.H.M. Date 12-2-39

STABILITY

Position of resultant : $\frac{16756}{1016.8} = 16.5'$; third pt. at $15.0'$

Bearing: $e = 6.0'$

$$b = \frac{1,016,800}{10 \times 45}\left(1 \pm \frac{6 \times 6}{45}\right) = 2260 \begin{cases} 1.80 = 4060 \text{ \#/□'} \\ 0.20 = \quad 450 \text{ \#/□'} \end{cases}$$

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I. **Page** 8

bject BIRGH HILL

nputation Gate tower

omputed by RSM **Checked by** R.H.M. **Date** 12-2-39

STABILITY - CASE II. - Water to spillway elevation, gates open.
This is the condition of maximum loading, except in extraordinary events. Concrete is designed for loads resulting from this assumption. Design is checked against loads resulting from water to surcharge elevation with allowable stresses increased by $\frac{1}{3}$.

	Force	Amt. (kips) ↓	↑	Amt. →	←	Arm	Moments abt. "A" ('kips) ↷	↶
	Structure	1412.3						32699
W1	$62.5 \times 10.4 \times 10 \times 13$	84.6				39.8		3370
W1a		101.0						4060
W2	$62.5 \times 6 \times 10.4 \times 22$	85.7				27.8		2380
W3		78.8						2050
W4a		69.0						525
U1	$62.5 \times 10 \times 45 \times 14.5$		408			22.5	9180	
U2	$62.5 \times 10 \times 45 \times 16/2$		225			30.0	6750	
U3	$62.5 \times 10 \times 10 \times 9/2$		28			41.7	1170	
PW1	$62.5 \times 10 \times 39.5^2/2$			488		13.2	6440	
PW2	$62.5 \times 10 \times 14.5^2/2$				65.7	4.8		315
		1831.4	661				23540	45399
	ΣV =	1170.4					ΣM =	21859

Position of resultant: $\frac{21859}{1170.4} = 18.7'$; third pt. at 15.0'.

$$e = \frac{45}{2} - 18.7 = 3.8'$$

Bearing: $\frac{1170400}{10 \times 45}\left(1 \pm \frac{6 \times 3.8}{45}\right) = 2600 \begin{cases} 1.51 = 3930 \#/\square' \\ 0.49 = 1270 \#/\square' \end{cases}$

WAR DEPARTMENT

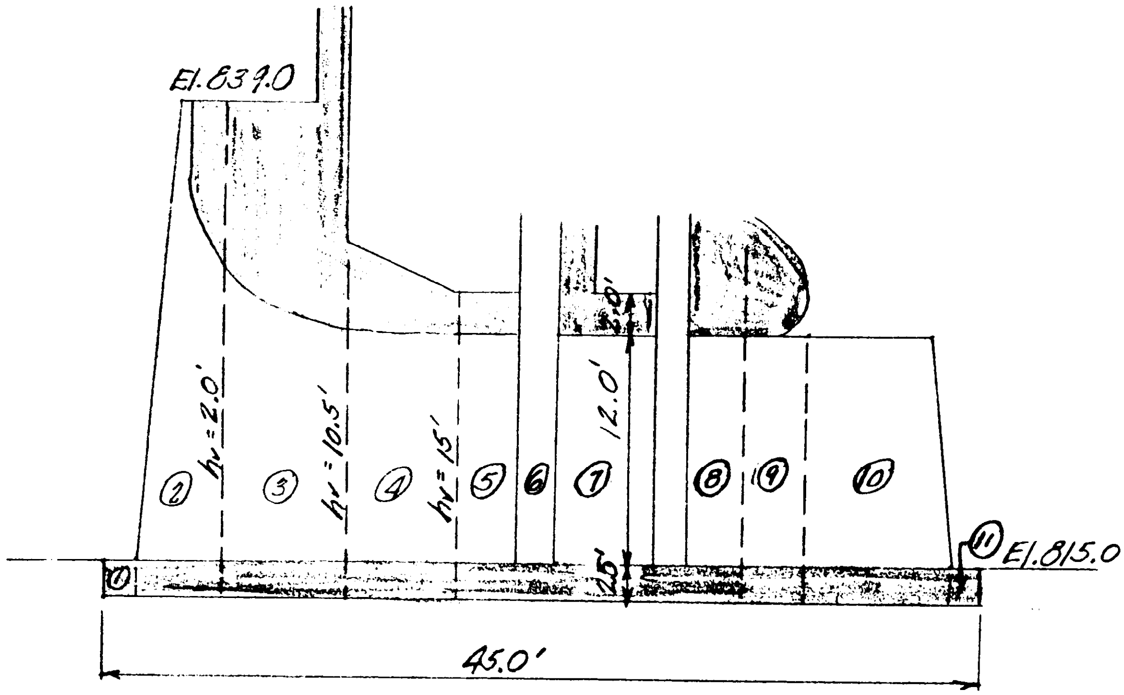
U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Subject: BIRCH HILL

Computation: Gate tower

Computed by: RSM Checked by: R.H.M. Date: 12-2-39

CONDUIT SECTION



Lateral pressure on pier exists with one gate open and adjacent passage closed and is equal to difference between spillway head and static head in open passage, or to approx. velocity head.

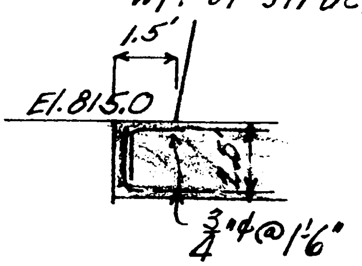
Section 1

Max. pressure with reservoir empty.

Wt. of structure = 1,412,300 / 10 x 45 = 3140 #/□'

Less floor slab -375

Net 2765 #/□'



$M = 2765 \times 1.5^2 / 2 = 3120$ '#

$A_s = 3180 \times 12 / 16000 \times .857 \times 24.5 = .11$ □"

Use 3/4"φ @ 1'-6"

$v = 2765 \times 1.5 / 12 \times .857 \times 24.5 = 17$ #/□"

bject BIRCH HILL
mputation Gate tower
mputed by PSM Checked by R.H.M. Date 12-2-39

CONDUIT SECTION

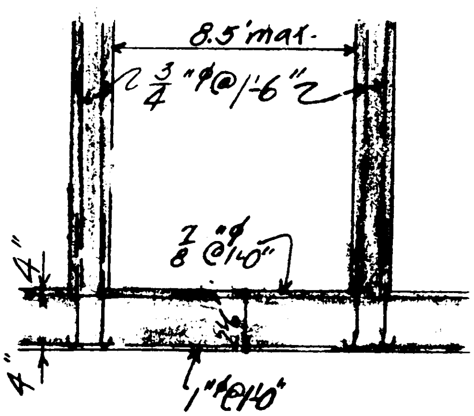
Section ②

Base slab - max. pressure with river down. Carries load from Sec.① in addition to own load

$M = \frac{2765 \times 5.5}{4} \times 8.5^2/12 = 22900$ #

$A_s = 22900 \times 12/16000 \times .857 \times 25.5 = .79$ □"/ft. Use 1"φ @ 1'-0"

$v = \frac{2765 \times 5.5}{4} \times \frac{8.5}{2} / 12 \times .857 \times 25.5 = 62$ #/□" Anchor negative steel



$u = \frac{16150}{3.14 \times .857 \times 25.5} = 237$ #/□"

Anchor negative steel.

$d = \sqrt{\frac{22900}{183.7}} = 11.9$" req'd. 25.5" supplied.

$f_c = \frac{2 \times 22900 \times 12}{.429 \times .857 \times 12 \times 25.5^2} = 191$ #/□"

Piers - pressure due to 2' differential head.

Span 17' ± d = 15" ±

$M = 125 \times 17^2/12 = 3020$ '#

$A_s = 3020 \times 12/16000 \times 15 \times .857 = .18$ □" Use 3/4"φ @ 1'-6"

$v = \frac{125 \times 17}{2} / 12 \times .857 \times 15 = 6.9$ #/□" OK

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I. Page 11

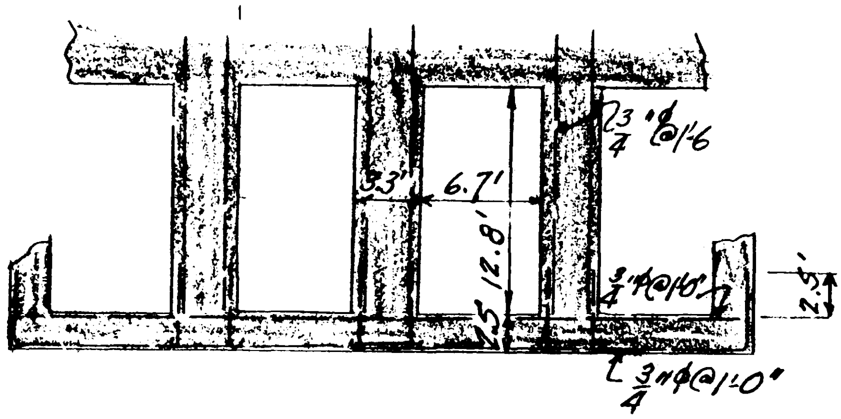
bject BIRCH HILL

mputation Gate tower

mputed by RSM Checked by R.H.M. Date 12-4-39

CONDUIT SECTION

Section ③ Reservoir full, 1 gate open.



Base slab - max. pressures due to wt. of structure, reservoir empty.

$M = 2765 \times \overline{6.7}^2/12 = 10{,}340'^{\#}$

$A_s = 10{,}340 \times 12/16000 \times .857 \times 25.5 = .35^{\square''}$. Use $\frac{3}{4}$"φ @ 1'-0" c.-c.

$V = 2765 \times 6.7/2 = 9260^{\#}$

$v = 9260/12 \times .857 \times 25.5 = 35\ ^{\#}/_{\square''}$ OK

$u = 9260/2.36 \times .857 \times 25.5 = 177\ ^{\#}/_{\square''}$. Anchor negative steel

Piers - lateral pressure of 10.5' hydrostatic head with one passageway open. Pier relatively stiffer than base slab, but lighter than roof, assume partially restraint at base, full restraint at roof.

$M = 62.5 \times 10.5 \times \overline{12.8}^2/10 = 10750'^{\#}$

$A_s = 10750 \times 12/16000 \times .857 \times 35 = .27^{\square''}$, Use $\frac{3}{4}$"φ @ 1'-6" c.-c.

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

bject BIRCH HILL

mputation Gate tower

mputed by RSM Checked by R.H.M. Date 12-4-39

CONDUIT SECTION

Section ④

1. Base - max. pressure due to wt. of structure, reservoir empty.

$M = 2765 \times \overline{6.5}^2/12 = \quad 9{,}700\ '\#$

$A_s = 9700 \times 12/16000 \times .857 \times 25.5 = .33^{\square''}$. Use $\frac{3}{4}''\phi$ @ 1'-0"

2. Pier. - lateral pressure due to hydrostatic head of 15'.

$M = wl^2/10 = 15 \times 62.5 \times \overline{12}^2/10 = \quad 13500\ '\#$

$A_s = 13500 \times 12/16000 \times .857 \times 37.5 = .32^{\square''}$ Use $\frac{3}{4}''\phi$ @ 1'-0"

$v = 938 \times 6/12 \times .857 \times 37.5 = 15\ \#/\square''$

Section ⑤

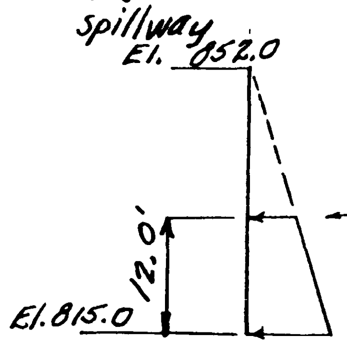
1. Base - Use $\frac{3}{4}''\phi$ @ 1'-0"

2. Pier

a. Lateral pressure due to hydrostatic pressure of 15'

$A_s = \frac{3}{4}''\phi$ @ 1'-0".

b. Load from gate slot section, emergency gate down, adjacent service gates closed.



Consider load distributed over 3' width

$62.5 \times 25 \times 3.3/2 = 2580\#$

$62.5 \times 37 \times 3.3/2 = 3820\#$

$M = \frac{2580}{3} \times \overline{12}^2/10 = \quad 12380\ '\#$

$\frac{1240}{2 \times 3} \times \overline{12}^2 \times .13 = \quad 3870$

$\quad 16250\ '\#$

$A_s = 16250 \times 12/16000 \times .857 \times 43.5 = .33^{\square''}$. Case "a" governs.

Subject: BIRCH HILL
Computation: Gate tower
Computed by: RSM Checked by: R.H.M. Date: 12-5-39

CONDUIT

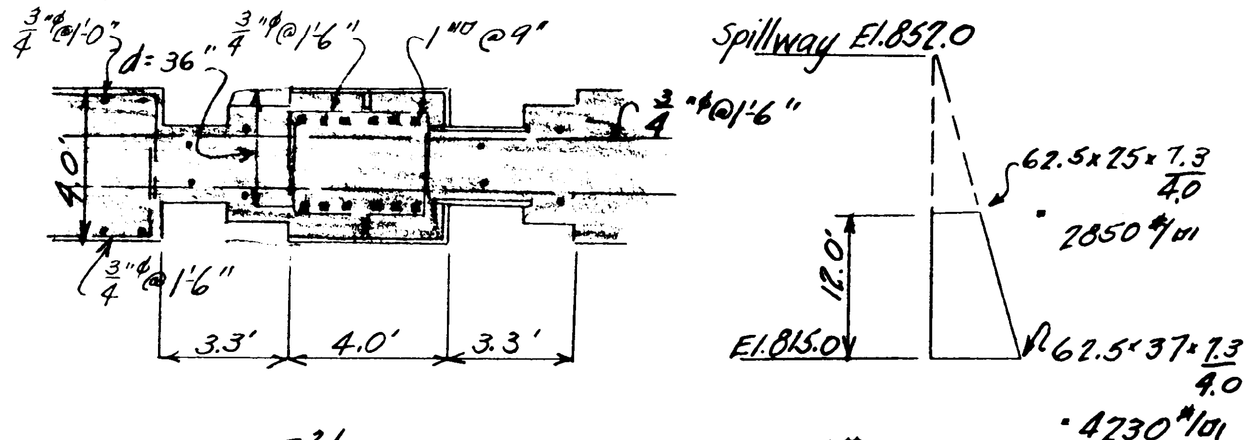
Sec. ⑥ Consider spanning slot horizontally. Span = 3.3'
With emergency gate down and adjacent passages closed, full hydrostatic pressure is developed on pier.

$P = 62.5 \times (852 - 815) = \quad 2310 \ \#/\square'$

$M = 2310 \times \overline{3.3}^2/12 \quad = 2090 \ '\#$

$A_s = 2090 \times 12/16000 \times .857 \times 19.5 = \ .10 \ \square''$. Use $\frac{3}{4}''\phi$ @ 1'6" c-c.

Sec. ⑦ Same condition as for Sec. ⑥. Pier section carries full hydrostatic pressure over section plus $\frac{1}{2}$ gate slot load.



$M = 2850 \times \overline{12}^2/10 = \qquad 41000 \ '\#$
$\quad \frac{1380 \times 12}{2} \times 12 \times .13 = \qquad \underline{12900}$
$\qquad\qquad\qquad\qquad\qquad 53900 \ '\#$

$A_s = 53900 \times 12/16000 \times .857 \times 36 = \ 1.31 \ \square''/ft$. Use 1"□ @ 9" c-c.

$V = 2850 \times \frac{12}{2} + \frac{1380 \times 12}{2} \times \frac{2}{3} = \ 22620 \ \#$

$v = 22620 / 12 \times .857 \times 36 = \quad 61 \ \#/\square''$ OK

$u = 22620 / 5.33 \times .857 \times 36 = \quad 138 \ \#/\square''$ OK

Base

Max. pressure, as before, 2765 #/□', net.
d eff. = 30 - 12 = 18"
$M = 2765 \times \overline{6}^2/12 = \quad 8295 \ '\#$
$A_s = 8295 \times 12/16000 \times .857 \times 18 = .42 \ \square''$. Use $\frac{3}{4}''\phi$ @ 1'0"

$v = 2765 \times 3/12 \times .857 \times 18 = \ 45 \ \#/\square''$ OK

CONDUIT

Sect. 8

Use $\frac{3}{4}$"φ @ 1'-0" in pier, vertically

Use $\frac{7}{8}$"φ @ 1'-0" in base, as for Sect. 9

Sect. 9

Pier Stress nominal. Use $\frac{3}{4}$"φ @ 1'-6", both directions

Base

Load down $12 \times 62.5 =$ 750 #/□'

$2.5 \times 150 =$ 375

1125 #/□'

Uplift 1060

Bearing 3400

4460 #/□'

Net loading 3335 #/□'

$M = 3335 \times 6^2/12 =$ 10,000'#

$A_s = 10{,}000 \times 12 / 16000 \times .857 \times 18 =$ .49 □"/ft.

Use $\frac{7}{8}$"φ @ 1'-0".

$V = 3335 \times 3 = 10000$#

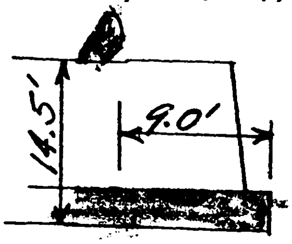
$v = 10000 / 12 \times .857 \times 18 =$ 54 #/□"

$u = 10000 / 2.75 \times .857 \times 18 = 238$ #/□" OK. Anchor negative steel.

Sec. 10

Pier Use $\frac{3}{4}$"φ @ 1'-6" both ways.

Test for shear at face of tower.



Edge of base

Load down (from Sec. 9) 1125 #/□'

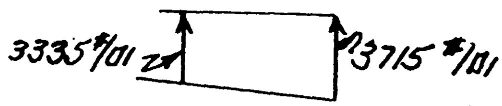
Load upward

Uplift 910

Bearing 3930

4840

Net 3715 #/□'



$v = 10 \times \frac{3715 + 3335}{2} \times 9 / 48 \times 174 =$ 38 #/□" OK

wever, the cost of railroad relocation for these sites would b

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I. — Page 16

ect BIRCH HILL

mputation Gate tower

mputed by RSM — Checked by R.H.M. — Date 12-5-39

CONDUIT

Sec. 10

Base

1. Pressure from Sec. 11 carried by downstream portion of base in this section, say, by a 3' strip.

$P = \frac{3715 + 3525}{2} \times 4.5/3.0 = 5430$ #/□' over 3' strip

$M = 5430 \times 7^2/12 = 22200$ '#

$A_s = 22200 \times 12 / 16000 \times .857 \times 25.5 = .76$ □" Use 1"□ @ 1'-0" c-c. in bottom, and $\frac{7}{8}$"φ @ 1'-0", top.

$V = \frac{5430 \times 7}{2} = 19000$ #

$v = 19000/12 \times .857 \times 25.5 = 72$ #/□". Anchor neg. steel.

$u = 19000/4 \times .857 \times 25.5 = 216$ #/□".

$f_c = \frac{2 \times 22200 \times 12}{.429 \times .857 \times 12 \times 25.5^2} = 189$ #/□"

2. Base, upstream portion

$M = 3525 \times 6.5^2/12 = 12400$ '#

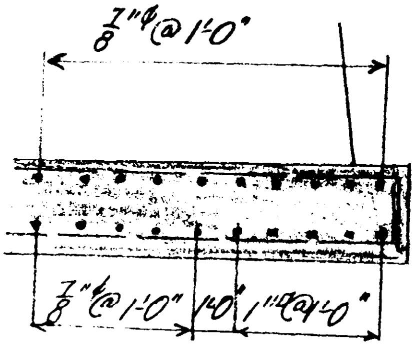
$A_s = 12400 \times 12/16000 \times .857 \times 25.5 = .425$ □".

Use $\frac{7}{8}$"φ @ 1'-0", top and bottom.

Sec. 11

$M = \frac{3930 + 3900}{2} \times 1.5^2/2 = 4400$ '#

$A_s = 4400 \times 12 / 16000 \times .857 \times 25.5 = .15$ □" Use $\frac{3}{4}$"φ @ 1'-6"



WAR DEPARTMENT

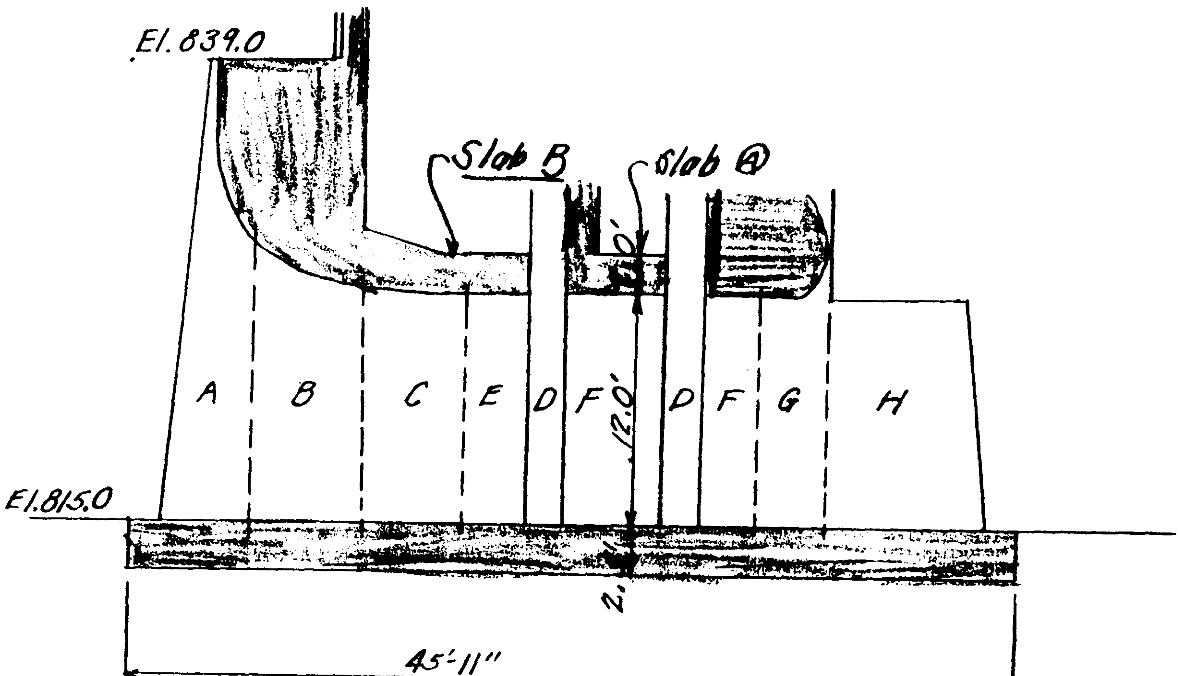
U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

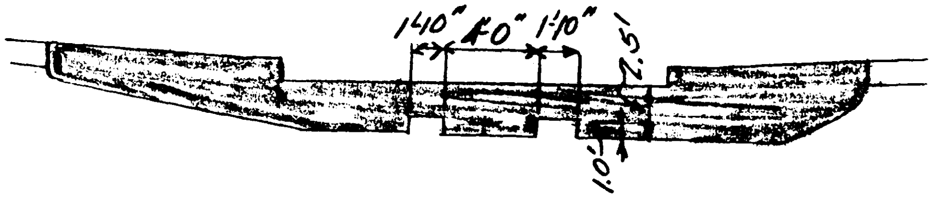
ject BIRCH HILL

mputation Gate tower -

mputed by RW Checked by RSM Date 12-6-39

CONDUIT SIDE WALLS





Hydrostatic head due to water to spillway el. exerted against sidewalls, considered effective over 50% of concrete area only due to adhesion of concrete to rock.

Slab A With gates closed, full hydrostatic head is exerted on this slab.

$P = (852 - 827) \times 62.5 = 1560\ \#/\square'$

$M = 1560 \times 6^2/12 = 4680'\#$

$A_s = 4680 \times 12 / 16000 \times .857 \times 12 = .34\ \square''$ Use $\frac{3}{4}''\phi$ @ 1'-0"

Slab B Pressures are same on both sides

Use $\frac{3}{4}''\phi$ @ 1'-6" c-c.

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page 17

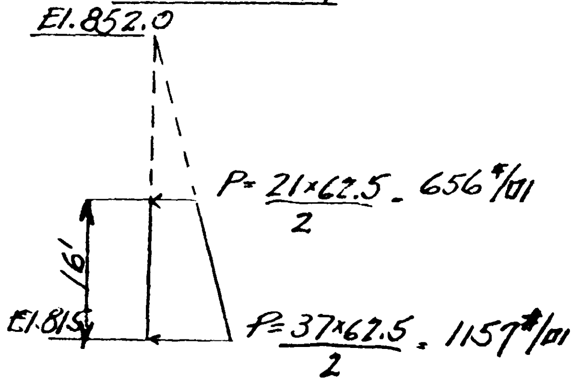
ject BIRCH HILL

mputation Gate tower

mputed by RW Checked by RSM Date 12-6-39

CONDUIT SIDE WALLS

Section A



$M = \frac{wl^2}{12} + \frac{w_1 l^2}{2 \times 10}$

$= \frac{656 \times \overline{16}^2}{12} + \frac{500 \times \overline{16}^2}{2 \times 10} = 20{,}340'^{\#}$

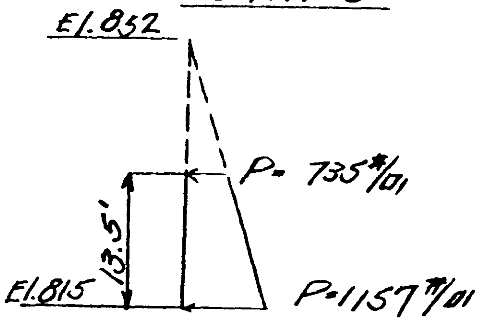
$A_s = \frac{20{,}340 \times 12}{16{,}000 \times .857 \times 25.5} = .70^{\square''}$

Use 1"ϕ @ 1'-0" c-c.

$V = \frac{656 \times 16}{2} + \frac{500 \times 16}{2} \times \frac{2}{3} = 7900^{\#}$

$v = 7900 / 12 \times .857 \times 25.5 = 30^{\#}/\square''$

Section B



$M = \frac{735 \times \overline{13.5}^2}{12} + \frac{422 \times \overline{13.5}^2}{20} = 15000'^{\#}$

$A_s = \frac{15000 \times 12}{16000 \times .857 \times 25.5} = .515^{\square''}$

Use 1"ϕ @ 1'-0"

$V = \frac{735 \times 13.5}{2} + \frac{422 \times 13.5}{2} \times \frac{2}{3} = 6860^{\#}$

$v = 6860 / 12 \times .857 \times 25.5 = 26^{\#}/\square''$

Section C

Use same steel as for Sec. B

Section D

Assume load on gate slot transferred to heavier sections on either side. Span = 1'-10"

Pressure on bottom 1ft strip $= \frac{852-815}{2} \times 62.5 = 1157^{\#}/\square'$

$1157 \times \overline{1.83}^2 / 12 = 3240'^{\#}$

$A_s = 3240 \times 12 / 16000 \times .857 \times 14 = .20^{\square''}$. Use $\frac{3}{4}$"ϕ @ 1'-6" c-c.

WAR DEPARTMENT

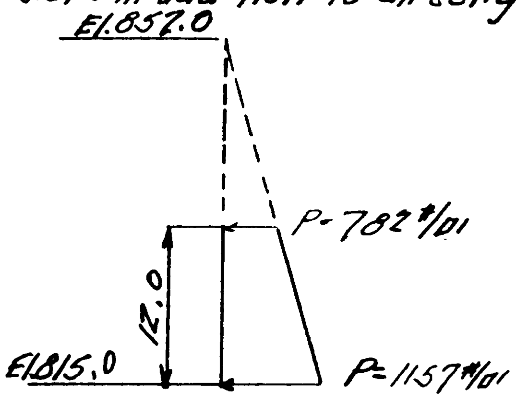
U. S. ENGINEER OFFICE, PROVIDENCE, R. I. **Page** 18

ject BIRCH HILL
nputation Gate tower
nputed by RW **Checked by** RSM **Date** 12-6-39

CONDUIT SIDE WALLS

Sect. F

Assume this section carries load as distributed from gate slots in addition to directly applied hydrostatic pressure.



$M = \left(\frac{782 \times \overline{12}^2}{12} + \frac{375 \times \overline{12}^2}{20}\right) \times \frac{5.8}{4} = 17500'^{\#}$

$A_s = \frac{17500 \times 12}{16000 \times .857 \times 18} = .85\,\square''$

Use 1"□ @ 1'-0" c-c.

$V = \left(\frac{782 \times 12}{2} + \frac{375 \times 12}{2} \times \frac{2}{3}\right) \times \frac{5.8}{4} = 9000^{\#}$

$v = 9000/12 \times .857 \times 18 = 48.5\,^{\#}/\square''$ OK

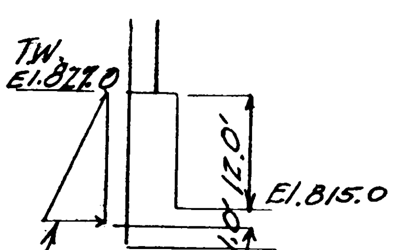
$f_c = \frac{2M}{kjbd^2} = \frac{2 \times 17500 \times 12}{.429 \times .857 \times 48 \times \overline{18}^2} = 73\,^{\#}/\square''$ OK

Section E Use loading & distribution as for Sec. F

$A_s = \frac{17500 \times 12}{16000 \times .857 \times 25.5} = .60\,\square''$ Use 1"φ @ 1'-0"

Sec. G stresses less than Sec. E. Use 1"φ @ 1'-0.

Sec. H



$P = 13 \times 62.5/2 = 406\,^{\#}/\square'$

Assume section cantilevered from base, and hydrostatic pressure due to water at TW El. 827.0

$M = 406 \times \overline{13}^2/6 = 11400'^{\#}$

$A_s = \frac{11,400 \times 12}{16000 \times .857 \times 25.5} = .38\,\square''$ Use $\frac{3}{4}$"φ @ 1'-0"

$V = 375 \times 12/2 = 2250^{\#}$

$v = \frac{2250}{12 \times .857 \times 25.5} = 9\,^{\#}/\square''$ OK

WAR DEPARTMENT

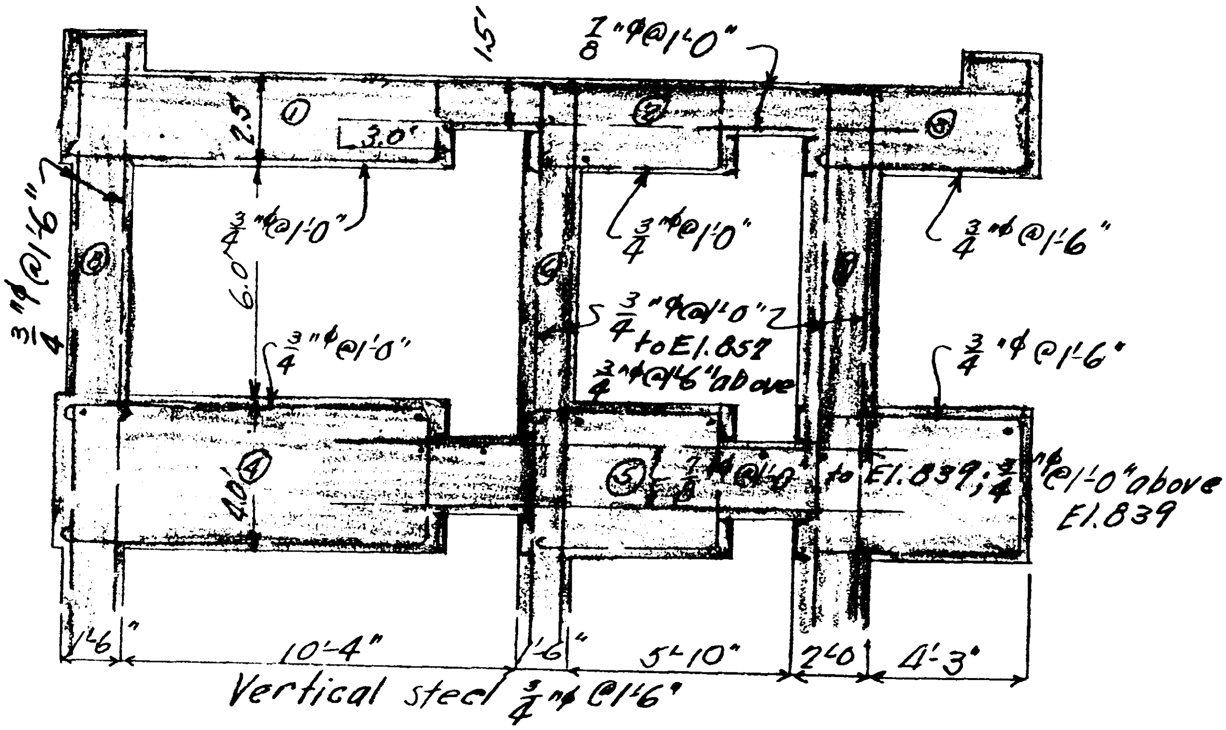
U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

ject BIRCH HILL

mputation Gate tower

mputed by RW Checked by RSM Date 12-6-39

GATE WELLS



Wall ①

Span 10'-4" Head $\frac{852-829}{2} = 11.5'$

$P = 11.5 \times 62.5 = 720\#/\square'$

$M = 720 \times \overline{10.3}^2/10 = 7700'\#$

$A_s = \frac{7700 \times 12}{16000 \times .857 \times 11} = .59\square''$ Use $\frac{7}{8}''\phi$@1'-0" in gate slot area.

$A_s = \frac{7700 \times 12}{16000 \times .857 \times 25.5} = .26\square''$ Use $\frac{3}{4}''\phi$@1'-0" in full section

$v = 7700 \times 10.3/2 \times 12 \times .857 \times 12 = 32\#/\square''$ OK

Wall ②

Span 5'-10" Head $(852-829)/2 = 11.5$.
Use steel as designed for Wall ①

Wall ③ Consider as cantilevered from Wall ⑥

$M = 720 \times \overline{4.3}^2/2 = 6650'\#$

$A_s = 6650 \times 12/16000 \times .857 \times 25.5 = .23\square''$ Use $\frac{3}{4}''\phi$@1'-6"

$v = 720 \times 4.3/12 \times .857 \times 25.5 = 12\#/\square''$

ject BIRCH HILL
mputation Gate tower
mputed by RW Checked by RSM Date 12-6-39

GATE WELLS

Wall ④

Design for conditions produced with passage open and adjacent passage closed. Difference in pressure is difference between spillway head and static head with gate open, or approx. velocity head.

$H_v = 15'$; $P = 15 \times 62.5 = 940$ #/□'

$M = 940 \times \overline{10.33}^2/10 = 10100$ '#

$A_s = 10100 \times 12/16000 \times .857 \times 17 = .53$ □" Use 7/8"φ @ 1'-0" thru gate slot.

$A_s = 10{,}100 \times 12/16000 \times .857 \times 43.5 = .21$ □" Use 3/4"φ @ 1'-0" in full section

$v = \frac{940 \times 10.33}{2} / 12 \times .857 \times 19.5 = 24$ #/□" OK

Wall ⑤

Water is sealed out of well with gate closed.
Condition as for Wall ④ exists under similar condition
See diagram for steel.

Wall ⑥

Design for full hydrostatic head

a Sec. at El. 829.0 Span = 6'

$P = (852-829) \times 62.5 = 1440$ #/□'

$M = \frac{wl^2}{12} = 1440 \times \overline{6}^2/12 = 4320$ '#

$A_s = 4320 \times 12/16000 \times .857 \times 13.5 = .28$ □" Use 3/4"φ @ 1'-0"

$v = 1440 \times 3/12 \times .857 \times 13.5 = 31$ #/□" OK

$f_c = 2 \times 4320 \times 12/.429 \times .857 \times 12 \times \overline{13.5}^2 = 120$ #/□" OK

b Sect. at El. 839.0

$M = (852-839) \times 62.5 \times \overline{8}^2/12 = 4350$ '#

$A_s = 4350 \times 12/16000 \times .857 \times 13.5 = .29$ □" Use 3/4"φ @ 1'-0"

Wall ⑦ No hydrostatic pressure exerted on this wall, but use 3/4"φ @ 1'-0" horizontal steel.

Wall ⑧ Condition as for Wall ④

$M = 940 \times \overline{6}^2/12 = 2820$ '#

$A_s = 2820 \times 12/16000 \times .857 \times 13.5 = .18$ □". Use 3/4"φ @ 1'-6"

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

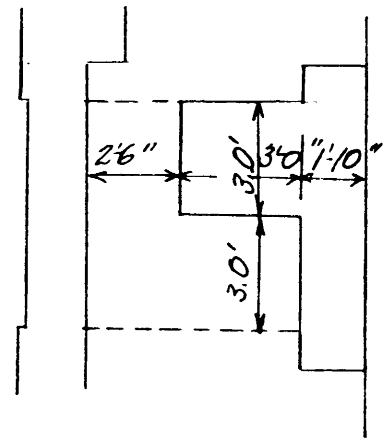
ject BIRCH HILL

mputation Gate tower

mputed by RSM Checked by Date 12-6-39

BASEMENT FLOOR -slab

Live load 350#/□'
Dead load
slab 150#/□'
Total 500#/□'



a Continuous section

$M = 500 \times \overline{6}^2/12 = 1500'^{\#}$

$A_s = \dfrac{1500 \times 12}{16000 \times .857 \times 9.5} = .14^{\square''}$

Use $\frac{5}{8}$"φ@1'-0", top & bottom.

$v = \dfrac{500 \times 6}{2} / 12 \times .857 \times 9.5 = 15.5^{\#}/\square''$

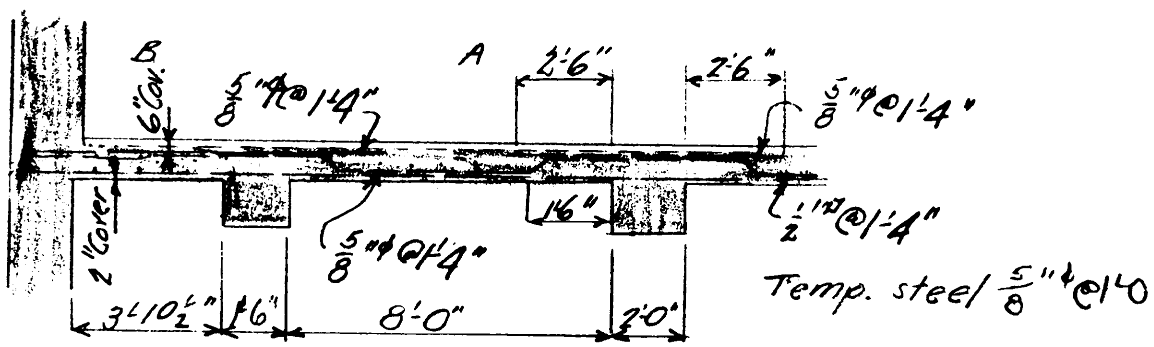
b Cantilever section

$M = 500 \times \overline{3}^2/2 = 2250'^{\#}$

$A_s = \dfrac{2250 \times 12}{16000 \times .857 \times 9.5} = .21^{\square''}$ Use $\frac{5}{8}$"φ @1'-0", top & bott.

OPERATING FLOOR

1. Slab - 8" slab - 4" cover



Panel A

$M = \dfrac{wl^2}{12} = 500 \times \overline{8}^2/12 = 2660'^{\#}$

$A_s = 2660 \times 12 / 16000 \times .857 \times 5.5 = .42^{\square''}$ Use $\frac{5}{8}$"φ@8" c-c.

$v = 500 \times 8 / 2 \times 12 \times .857 \times 5.5 = 35.4^{\#}/\square''$ OK

Panel B

$M = wl^2/12 = 500 \times \overline{3.78}^2/12 = 600'^{\#}$

$A_s = 600 \times 12 / 16000 \times .857 \times 5.5 = .10^{\square''}$. Use $\frac{5}{8}$"φ@1'-4"

ject BIRCH HILL

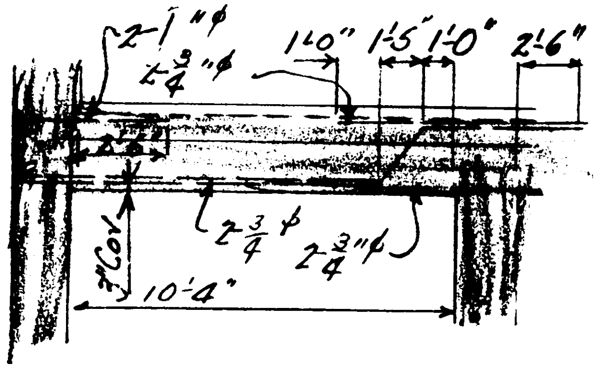
mputation Gate tower

mputed by RSM Checked by Date 12-11-39

OPERATING FLOOR

Beam A

Loading - 280#/□' Assume uniform over area.
Assume beam fixed at one end, partially restrained at other



$L = 280 \times 10 = 2800$ #/lin. ft.

$M = 2800 \times \overline{10.4}^2 \times \frac{1}{8} = 38000$ '#

$A_s = \frac{38000 \times 12}{16000 \times .857 \times 20.5} = 1.62$ □"

Use 4-3/4"φ

$V = 2800 \times 10.4 \times \frac{5}{8} = 18,000$ #

$v = 18,000/24 \times .857 \times 20.5 = 43$ #/□" OK

Beam B

Loading - as for Beam "A"

$L = 280 \times 7.7 = 2150$ #/lin. ft.

$M = 2150 \times \overline{10.4}^2 \times \frac{1}{8} = 29000$ '#

$A_s = 29000 \times 12/16000 \times .857 \times 20.5 = 1.24$ □"

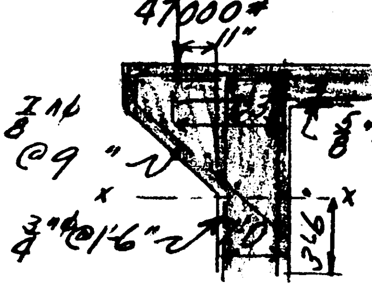
Use 2-3/4"φ bent, 2-5/8"φ, straight, at fixed end
Use 2-7/8"φ, straight, top, at free end.

Arrange as for Beam A

$V = 2150 \times 10.4 \times .63 = 14000$ #

$v = 14000/18 \times .857 \times 20.5 = 44$ #/□"

GATE STAND SUPPORT



Assume 3' distribution of load

$A_s = \frac{47000 \times 11}{3 \times 16000 \times .857 \times 19.5} = .65$ □" Use 7/8"φ @9"

$v = \frac{47000}{3 \times 12 \times .857 \times 19.5} = 78$ #/□" across wall at "x"

DESIGN FOR SURCHARGE ELEVATION

For a check against stresses resulting from hydrostatic loading to surcharge elevation, representative sections have been chosen and redesigned using stresses increased 33⅓ %. With water to spillway surcharge elevation, tailwater is at El. 842.0, and stresses are actually less than for conditions with water to spillway elevation and gates closed. The two sections redesigned are sections giving severest stresses for design condition and are reinvestigated for effect of increased water loading.

1. From pg. 10 - Section ②, base slab.
Design for full reservoir.

Water	44 × 62.5	2750 #/□'
Concrete	2.5 × 150	375
		3125 #/□'
Bearing	650	
Uplift	2840	
		3490
Net		365 #/□'

$A_s = 365 \times 12 / 21{,}300 \times .857 \times 25.5 = .10\square''$, against $.79\square''$ for design head.

Section OK for higher head.

2. From pg. 20 - Wall ⑥ a - No water in gate well.
Sec. at El. 829.0. Span = 6'

$P = (859-829) \times 62.5 = 1875$ #/□'

$M = 1875 \times \bar{6}^2/12 = 5625'\#.$

$A_s = 5625 \times 12/21{,}300 \times .857 \times 13.5 = .27\square''$, against $.28\square''$ for design head

$f_c = 2 \times 5625 \times 12/.429 \times .857 \times 12 \times 13.5^2 = 166$ #/□" Actual
1333 #/□" Allowable

Section OK.

WAR DEPARTMENT

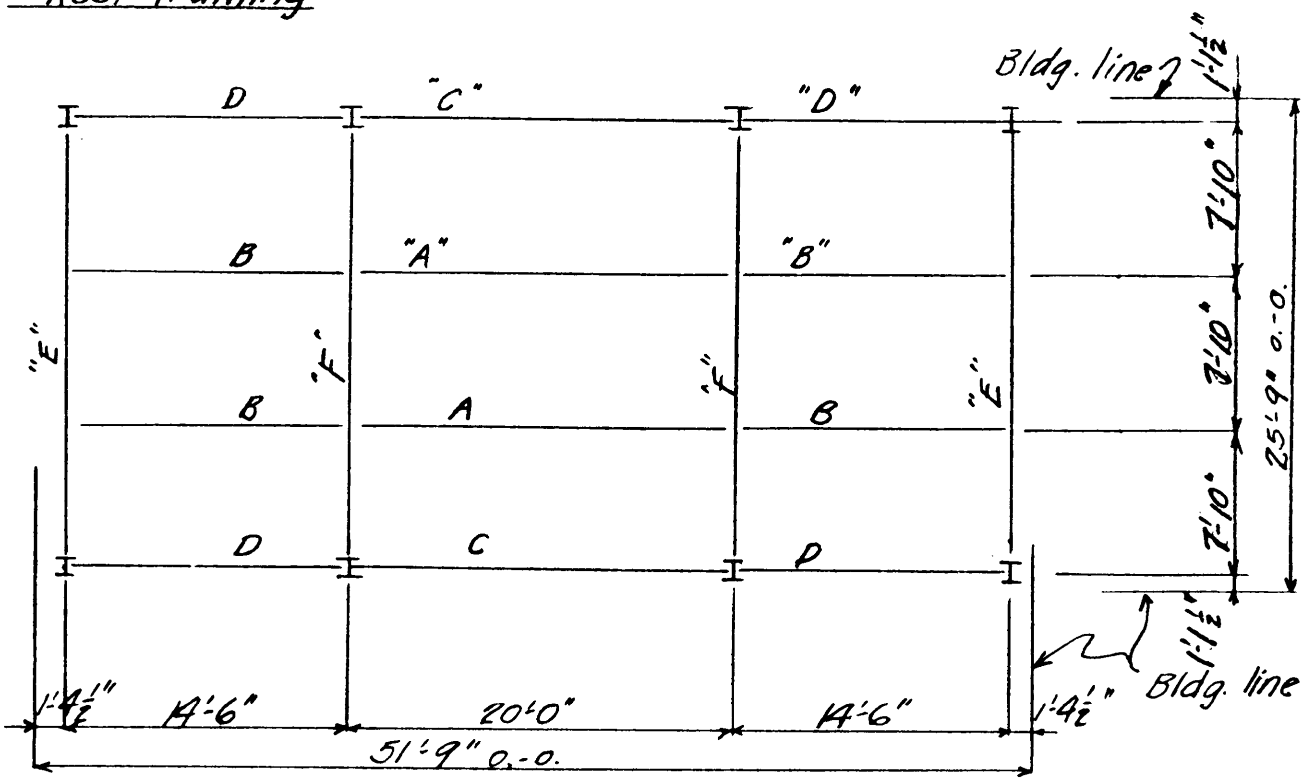
U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

ject BIRCH HILL

nputation Operating house - structural steel

nputed by RSM Checked by R.H.M. Date 12-4-39

Roof framing



Roof will consist of a flat reinforced concrete slab with a minimum of 1" of sawdust concrete - sloped ½" per foot for drainage plus a 5-ply tar and gravel roofing.

Assume average thickness of sawdust concrete to be 5", weighing 45#/□'. Tar and gravel roofing = 5#/□'.

Assume a snow load of 40#/□'.

Object: BIRCH HILL
Computation: Operating house - structural steel
Computed by: RSM Checked by: R.H.M. Date: 12-4-39

Roof slab

Loading

	snow	40 #/□'
	roofing	5
	sawdust conc.	45
	roof slab (5")	63
		153 #/□'

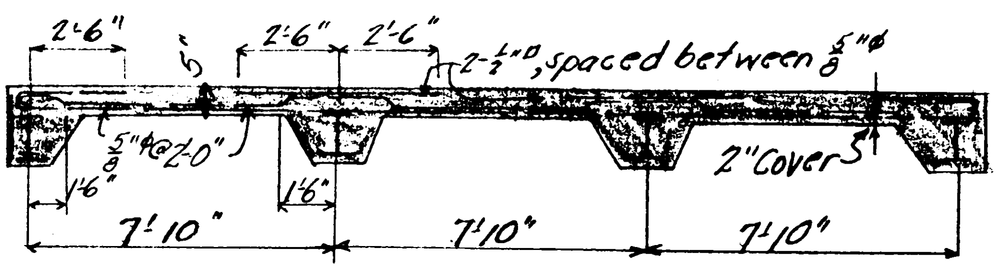
$-M = wl^2/8 = 153 \times \overline{7.8}^2/8 =$ 1160 '#

$+M = wl^2/10 = 153 \times \overline{7.8}^2/10 =$ 930 '#

$d = \sqrt{\frac{1160}{183.7}} =$ 2.6" reqd., 3" supplied

$-A_s = \frac{1160 \times 12}{16000 \times .857 \times 3} =$ .36 □". Use $1-\frac{5}{8}$"φ & $2-\frac{1}{2}$"φ for each 2' strip

$+A_s = \frac{930 \times 12}{16000 \times .857 \times 3} =$ .29 □". Use $\frac{5}{8}$"φ @ 1'-0", bend up alt. bars.



Roof purlins - "A"

Loading -

	roof slab	$8 \times 153 =$	1224 #/lin. ft.
	beam		40
	conc. encasement		226
			1490 #/lin. ft.

$M = 1490 \times \overline{20}^2 \times 12/8 =$ 896000 "#

$S = 896{,}000/18{,}000 =$ 49.7. Use 12 WF 40 $S = 51.9$

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

bject BIRCH HILL

mputation Operating house - structural steel

mputed by RSM Checked by RHM Date 12-4-39

Purlins - B

$M = 1490 \times \overline{14.5}^2 \times 12/8 = 470{,}000''\#$

$S = 470{,}000/18{,}000 = 26.1$ Use 12 WF 25 $S = 30.9$

Purlins - C

Loading - slab $4 \times 153 =$ $612\ \#/ft.$

parapet $4 \times 1 \times 130 =$ 520

beam 40

encasement 218

$1390\ \#/ft.$

$M = 1390 \times \overline{20}^2 \times 12/8 = 841{,}000''\#$

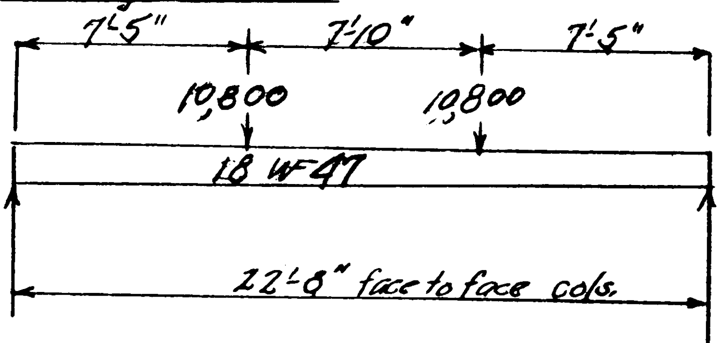
$S = 841{,}000/18000 = 46.6$ Use 12 I 45, $S = 47.3$

Purlins - D

$M = 1390 \times \overline{14.5}^2 \times 12/8 = 439{,}000''\#$

$S = 439{,}000/18{,}000 = 24.4$ Use 12 I 31.8. $S = 36.0$

Roof girder - E



Assume 18 WF 47

$M = [10800 \times (11.33 - 3.83)] \times 12 =$ 970,000 "#

$47 \times 22.7^2 \times 12/8 =$ 36,000

1,006,000 "#

$S = 1{,}006{,}000/18000 = 56.0$

S for 18 WF 47 = 82.3

$V = 10{,}800 + 47 \times 12.4 = 11{,}400\#$

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

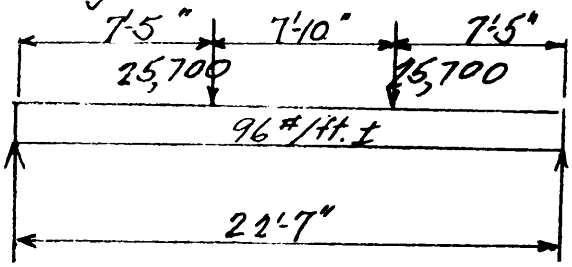
Page 27

ject BIRCH HILL

mputation Operating house - structural steel

mputed by RSM Checked by R.H.M. Date 12-4-39

Roof girder "F"



Assume 18 WF 96

$M = 25700 \times 7.4 \times 12 =$ 2,290000 "#

$96 \times 22.7^2 \times 12/8 =$ 75000

2,365,000 "#

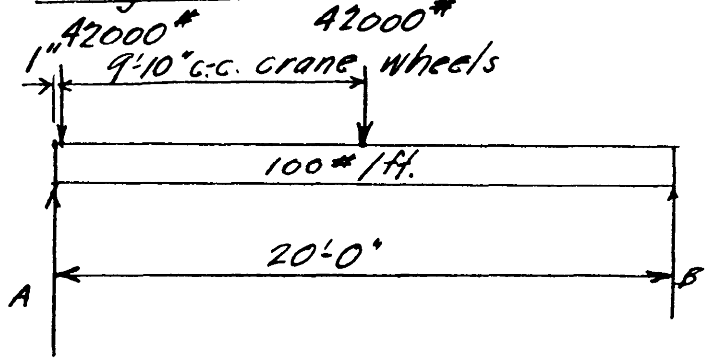
$S = 2365{,}000/18000 = 131.0$

Use 18 WF 77 $S = 141.7$

$V = 25{,}700 + 96 \times 11.4 =$ 27,000 #

Project: BIRCH HILL
Computation: Operating house - structural steel
Computed by: RSM Checked by: R.H.M. Date: 12-4-39

Crane girder



20-T. crane girder, hand operated.
Load 42000# on each wheel, occurring on breaking gate seal.
Side thrust - carrying wt. of gate = 14000 × 10% = 1400#, distributed 350# to each wheel.

$R_A = 42000(10.1 + 19.9)/20 = 63000\#$
$100 \times 10 = 1000$
$64000\#$

$M = [64000 \times 9.9 - (42000 \times 9.9 + 1000 \times 5)] \times 12 = 2,560,000''\#$

Assume 24 WF 74.

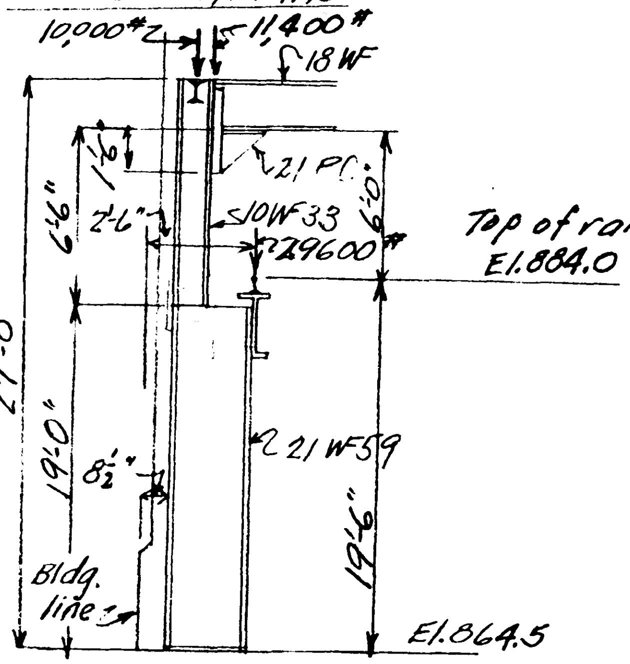
f_s (actual) = $2560,000/170.4 =$ 15,000 #/in

f_s(allowable) $= \dfrac{20000}{1 + \dfrac{l^2}{2000b^2}} = \dfrac{20000}{1 + \dfrac{240^2}{2000 \times 9^2}} =$ 14,750

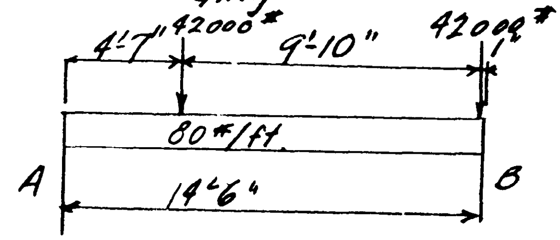
Use 24 WF 80, both spans.

Project BIRCH HILL
Computation Operating house - structural steel
Computed by RSM Checked by R.H.M. Date 12-4-39

End columns



Crane loading



$R_A = 80 \times 7.25 =$ 580#
$42000 \times \frac{9.9 + .1}{14.5} =$ 29020
Total 29600#

Wind load - taken by end wall.

Upper section Assume 10WF21 Use 10WF33 to get 5⁄₁₆ ga.

Actual stress
Direct $21,400 / 6.19 =$ 3460 #/□".
Eccentric $11,400 \times 5 / 21.5 =$ 2650
Total 6110 #/□".

Allowable
$$\frac{18000}{1 + \frac{60^2}{18000 \times 4.14^2}} = 18000 \text{ \#/□"}$$ OK

Lower section
Assume 21 WF 59
Loading on inner flange
Crane loading 29,600#
$fs = 29600 / 8.23 \times .575 =$ 6,250#/□" OK
Loading on entire section
Actual
$52000 / 17.36 =$ 3,000#/□"
$10000 \times 5.5 =$ 55000
$11,400 \times .5 =$ 5700
$-29,600 \times 10.75 =$ -319700
$259,000 / 119.3 =$ 2160
5160 #/□"

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page 30

ject BIRCH HILL

mputation Operating house - structural steel

mputed by RSM Checked by RHM. Date 12-4-39

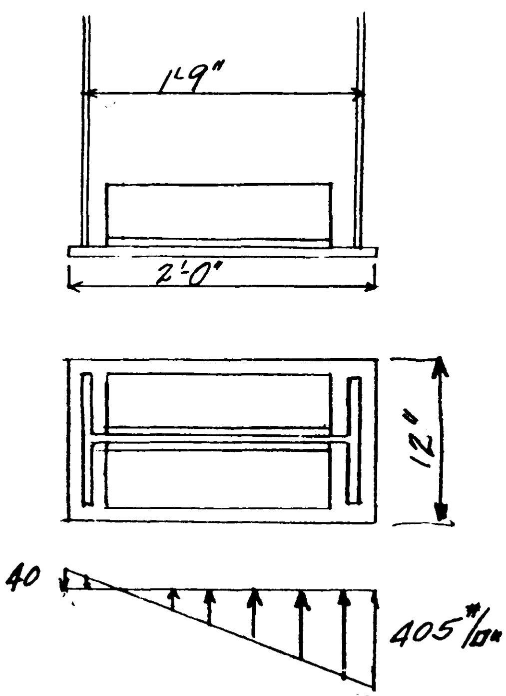
End col.

Lower sect.

Allowable stress

$$\frac{18000}{1+\frac{228^2}{18000\times1.68^2}} = \qquad 8,900\ \#/\square'' \text{ OK}$$

Col. base



$$f_c = \frac{P}{A} + \frac{6m}{bd^2} = \frac{52000}{24\times12} + \frac{6\times259000}{12\times24^2}$$

$$\left.\begin{array}{l} = 405\ \#/\square''\text{, actual} \\ = 625 \quad \text{, allowable} \end{array}\right\} \text{OK.}$$

Thickness $t = p\sqrt{\frac{3b'}{f}}$

$$= \left(\frac{12-.39}{2} - .5\right)\sqrt{\frac{3\times370}{18000}} = 1.3''$$

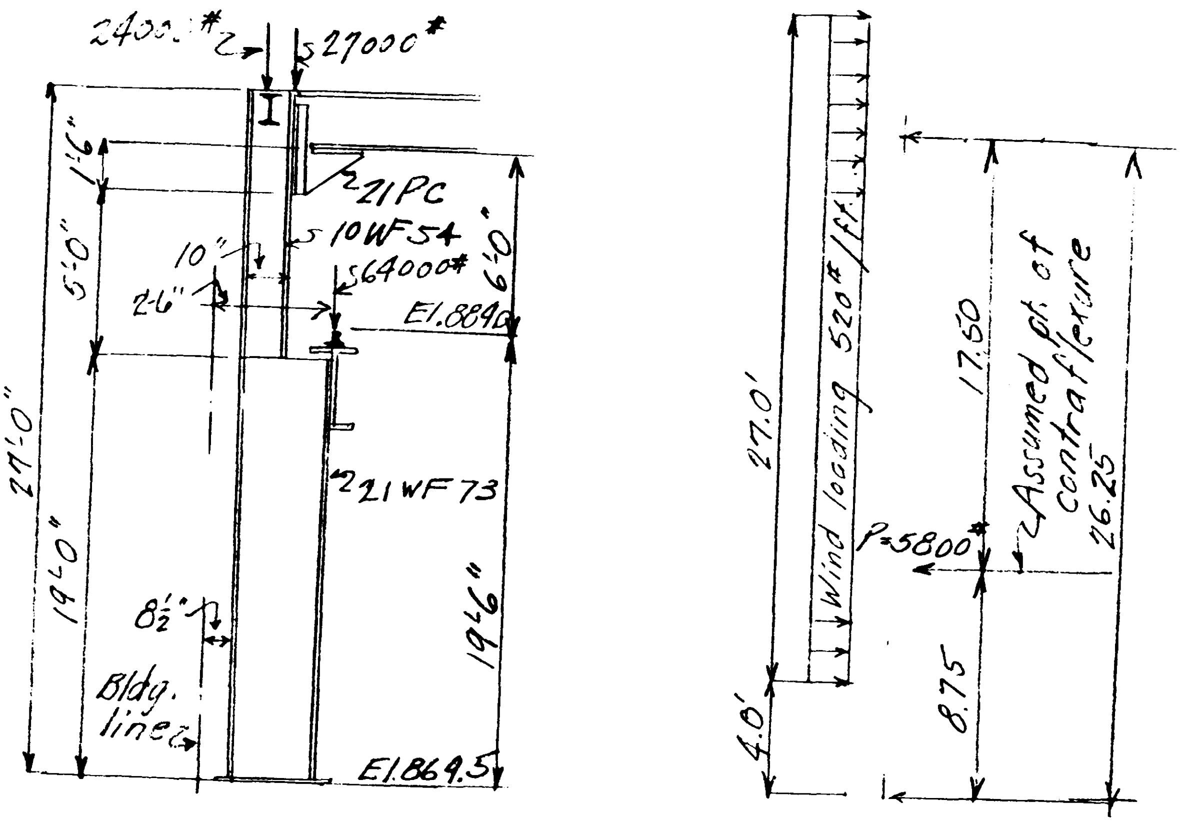
thickness of web L = .5"

$t = 1.3 - .5'' = .8''$ net

Use base plate 12" x 1" x 2'-0"

ject BIRCH HILL
omputation Oper. house - structural steel
omputed by RSM Checked by R.H.M. Date 12-4-39

Intermediate column



Wind loading -30#/□"
Load per ft. = $\frac{20+14.5}{2} \times 30 = 520$#

$P = [520 \times (4.75 + 17.50)] \div 2 =$ 5800# per column

Upper section
Assume 10 WF 54
Actual stress
Direct 51000 / 15.88 = 3210
Eccentricity 27000 × 5/60.4 = 2240
Wind - leeward cd.
5800 × 15.25 × 12 / 60.4 = 17600

23050 #/□"

Allowable stress

$$\frac{18000 \times 4/3}{1 + \frac{60^2}{18000 \times 4.39^2}} =$$

24000#/□" } OK

ject: BIRCH HILL

omputation: Oper. house- structural steel

omputed by: RSM Checked by: R.H.M Date: 12-5-39

Inter. col.

Lower section

Assume 21 WF 73

1. Loading on inner flange 64,000#

$f_s = 64000 / 8.295 \times .74 =$ 10,400 #/□"

Allowable loading

$$\frac{18000}{1 + \frac{(19 \times 12)^2}{18000 \times (.289 \times 8.295)^2}} = 12{,}000 \text{ #/□" OK}$$

2. Loading on entire section

Direct $117000 / 21.46 =$ 5450 #/□"

Ecc. $-24000 \times 5.62 = -134900$

$-27000 \times .62 = -16700$

$+64000 \times 11.01 = +705000$

$553400 / 150.7$ 3670

Wind $(5800 - 520 \times 8.75) \times 8.75 \times 12 = 131,300$

$520 \times 4.75 \times 7.15 \times 12 = 212000$

$343200 / 150.7 =$ 2280

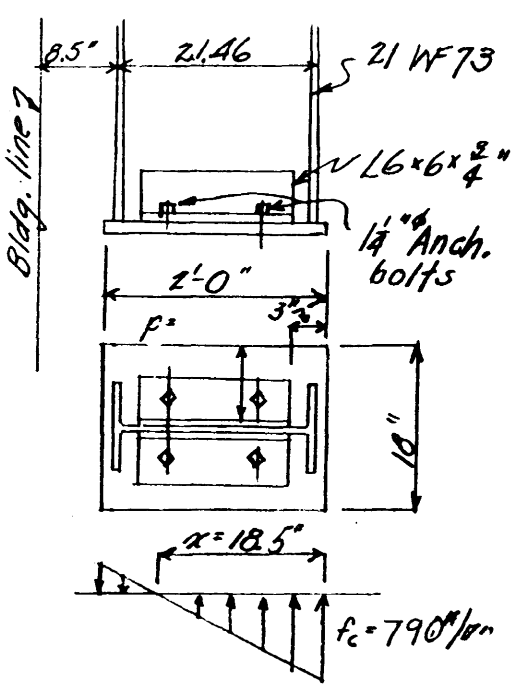
Total actual f_s 11400 #/□"

Allowable stress

$$\frac{18000 \times 4/3}{1 + \frac{(19 \times 12)^2}{18000 \times 1.76^2}} = 12{,}400 \text{ #/□"}$$

OK

Column base



Base plate $18 \times 1\frac{1}{2}" \times 24$

1. Size

$$f_c = \frac{P}{A} + \frac{6m}{bd^2} = \frac{117000}{18 \times 24} + \frac{6(343000 + 553000)}{18 \times 24^2}$$

$= 790$ #/□"

Allowable $f_c = 4/3 \times 625 = 833$ #/□" OK

2. Thickness

$$x = \frac{d}{2}\left(\frac{P \times d}{6m} + 1\right) = 12\left(\frac{117000 \times 24}{6 \times 896000} + 1\right) = 18.5"$$

Test at web L.

$$t = p\sqrt{3b'/f}$$

$$= \left(\frac{18 - .455}{2} - .75\right)\sqrt{\frac{3 \times 790 \times \frac{15.3}{18.5}}{24000}} = 2.33"$$

$2.33 - .75 = 1.58"$ plate thickness reqd.

Use PL $18 \times 1\frac{1}{2} \times 24"$

Web Ls $6 \times 6 \times \frac{3}{4}"$

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

ject BIRCH HILL Page 33

mputation Oper. house - structural steel

mputed by RSM Checked by R.H.M. Date 12-5-39

Wind-bracing cols., end walls.

Span 25'6"

Wind load $30 \times 12 = 360$ #/ft. of height.

Assume 10 WF 33

$l/b = 306/8 = 38.2$ OK

$M = 360 \times \overline{25.5}^2 \times 12/8 = 351,100$ "#

$S = 351,100 / 18,000 = 19.5$

S for 10 WF 33 = 35.0 } OK.

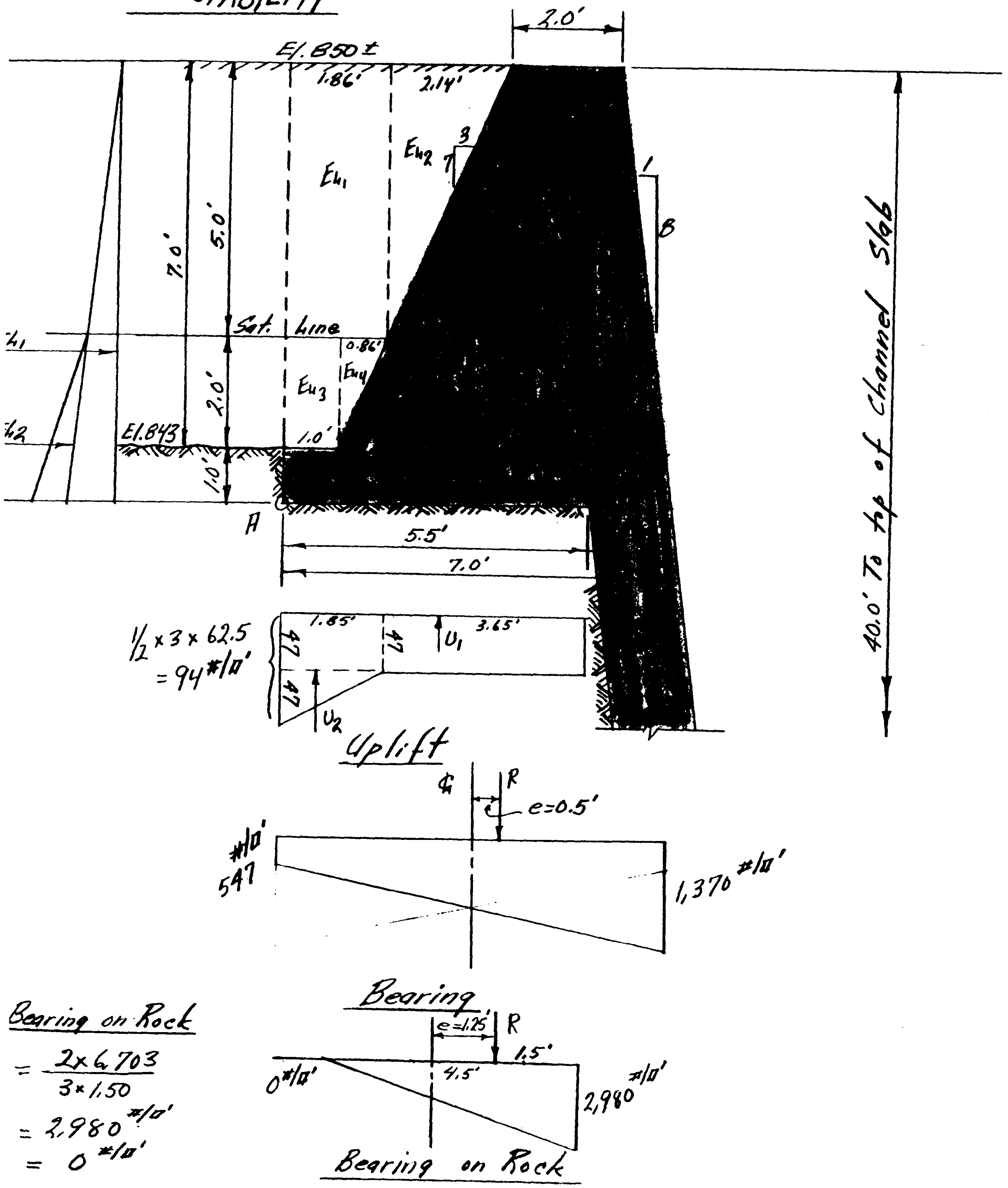
Deflection

$\Delta = k \frac{W l^3}{EI} = \frac{5}{384} \times \frac{(360 \times 25.5) \times \overline{306}^3}{29000000 \times 170.9} = .69"$

Δ (allowable) $= 306/360 = .85"$

Project Birch Hill Dam
Computation Outlet channel retaining wall
Computed by JM Checked by Date Nov. 9, 1939

STABILITY



Bearing on Rock

$= \frac{2 \times 6,703}{3 \times 1.50}$

$= 2,980$ #/□'

$= 0$ #/□'

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Subject: BIRCH HILL — Page 35

Computation: Retaining Wall

Computed by: VSM — Checked by: ______ — Date: Nov. 9, 1939

STABILITY

	Forces acting	↓	↑	→	←	Arm	Moments about A ↷	↶
C1	$150 \times \frac{1}{2} \times 3 \times 7$	1,575				3.0	4,725	
C2	$150 \times 7 \times 2$	2,100				5.0	10,500	
C3	$150 \times \frac{1}{2} \times .87 \times 7$	457				6.3	2870	
C4	$150 \times 1 \times 7$	1,050				3.5	3680	
E1	$100 \times 1.86 \times 5$	930				.9	865	
E2	$100 \times 2.14 \times \frac{1}{2} \times 5$	535				2.6	1,375	
E3	$125 \times 1 \times 2$	250				.5	125	
E4	$125 \times \frac{1}{2} \times .86 \times 2.0$	108				1.3	130	
U1	47×5.5		258			2.8		710
U2	$47 \times \frac{1}{2} \times 1.85$		44			.6		27
P1	$35 \times \frac{1}{2} \times 8^2$			1120		2.7	2990	
P2	$43 \times \frac{1}{2} \times 3^2$			203		1.0	203	
		7,005	302	1,323			27471	737
	$\Sigma V =$	6,703				$\Sigma M =$	26734	

Position of resultant $= \frac{\Sigma M}{\Sigma V} = \frac{26734}{6,703} = 4.0$; third pt. at 4.67'

Middle third of base resting on rock = 3.67'.

$e = 4.0 - \frac{7.0}{2} = .50$

$$\text{Bearing} = \frac{6.703}{7.0}\left(1 \pm \frac{6 \times .5}{7}\right) = \frac{6.703}{7} \times 1.43 = 1370 \ \#/\square' $$
$$\times .57 = 547 \ \#/\square'$$

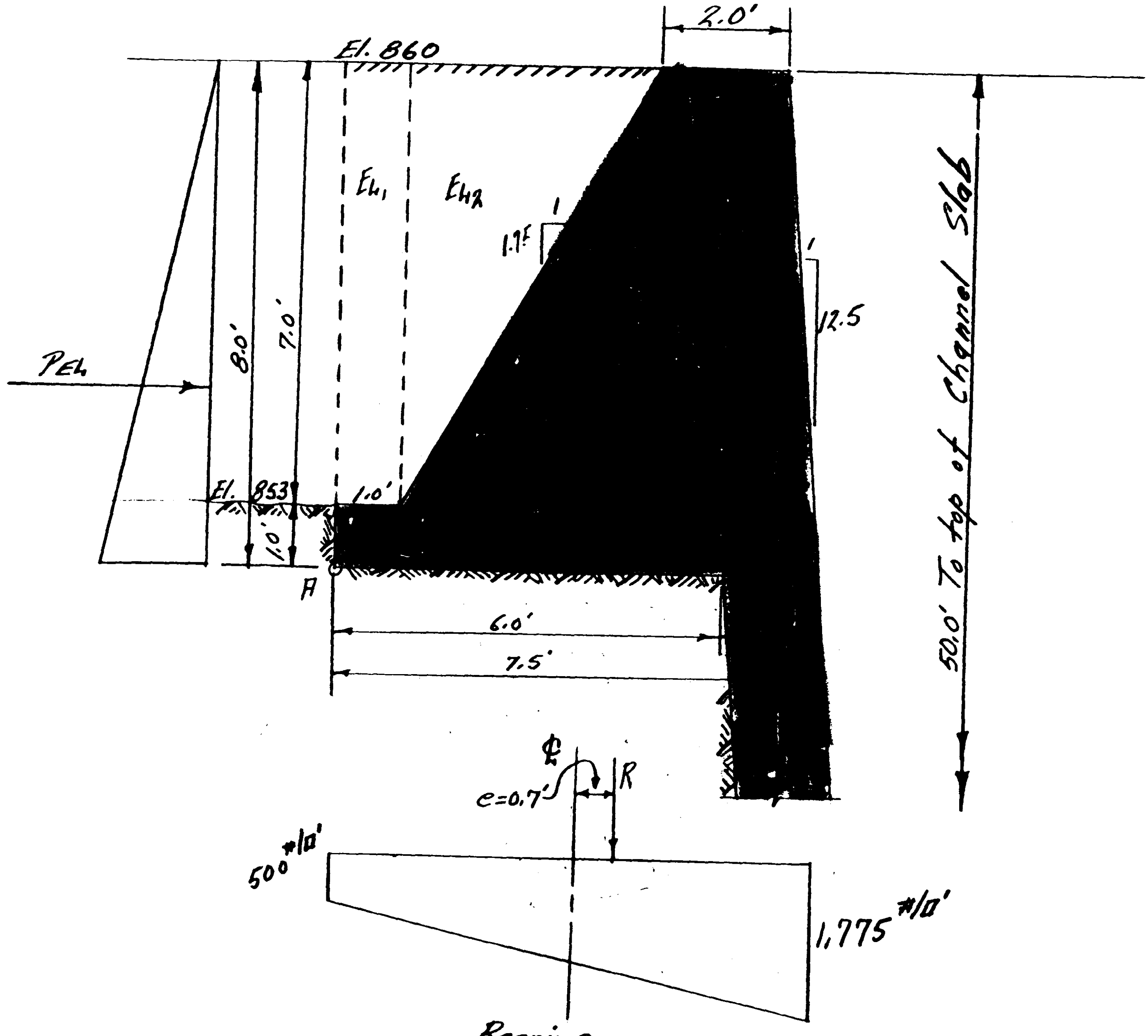
WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

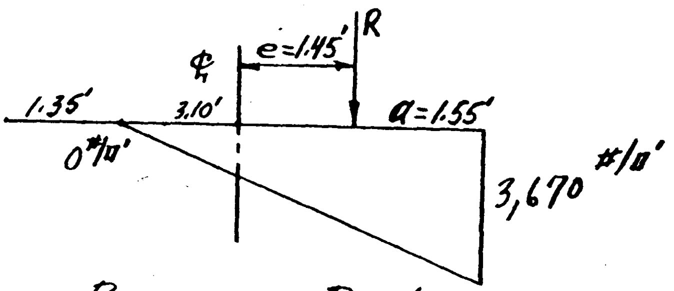
bject Birch Hill Dam

mputation Retaining Wall N.W. & S.W. of Operating House @ Sta. 12+25±

mputed by JRM Checked by ______ Date Nov. 9, 1939



Bearing



Bearing on Rock

Bearing on Rock

$$= \frac{2 \times 8,545}{3 \times 1.55}$$

$$= \begin{Bmatrix} 3,670 \ \#/\square' \\ 0 \ \#/\square' \end{Bmatrix}$$

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page 37

Subject BIRCH HILL

Computation Retaining wall

Computed by JSM Checked by Date Nov. 9, 1939

STABILITY

	Forces acting	↓	↑	→	←	Arm	Moments about ↷	↶
C1	150 × ½ × 4 × 7	2100				3.7	7700	
C2	150 × 7 × 2	2100				6.0	12600	
C3	150 × 7 × .56 × ½	1120				6.2	6930	
C4	150 × 7.5 × 1	1125				3.8	4220	
E1	100 × 7 × 1	700				.5	350	
E2	100 × 7 × 4 × ½	1400				2.3	3260	
PE	35 × ½ × 8.0²			1120		2.7	2990	
		8,545		1120			38050	
	ΣV =	8545				ΣM =	38050	

Position of resultant $= \frac{\Sigma M}{\Sigma V} = \frac{38050}{8545} = 4.45'$, third pt. at 5.00'

Third pt. of base resting on rock = 4.0'.

$e = 4.45 - 7.5/2 = .7'$

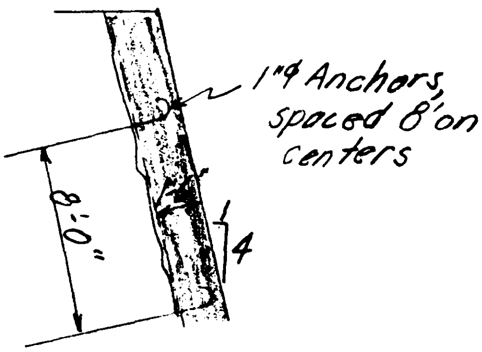
Bearing $= \frac{7.545}{7}\left(1 \pm \frac{6 \times .7}{7.5}\right) = 1775^{\#}/_{\square'}$ to $500^{\#}/_{\square'}$

... greater freedom from trouble due to ice and drift. etc.

bject BIRCH HILL

mputation Outlet channel lining

mputed by RSM Checked by ______ Date 1-2-40



Anchors are assumed effective over a width of 4', with gravel drains relieving hydrostatic pressure over the remaining 4' of spacing on centers.

Pull on one anchor, allowable $= 18000 \times .7854 = 14{,}100^{\#}$

Pull per sq. ft. of controlled area $= \frac{14100}{4 \times 8} = 440^{\#}$

Resisting wt. of concrete $= 1.5 \times 150 \times \frac{1}{4} =$ $\underline{56}$

Total resisting force, per sq. ft. $516^{\#}$

Consider 50% adhesion of concrete to rock.

Then anchors are able to resist a hydrostatic pressure through the rock of

$$\frac{516}{62.5/2} = \underline{\underline{16.5 \text{ ft.}}}$$

$8 \times 8 \times 1.5 \times 150^{\#} = 14{,}400$ shear $13{,}500^{\#}/\square''$

Area $\frac{14{,}400}{13{,}500} = 1.68^{\square''}$

Use $1\frac{1}{4}''\square$ Area $1.56^{\square''}$

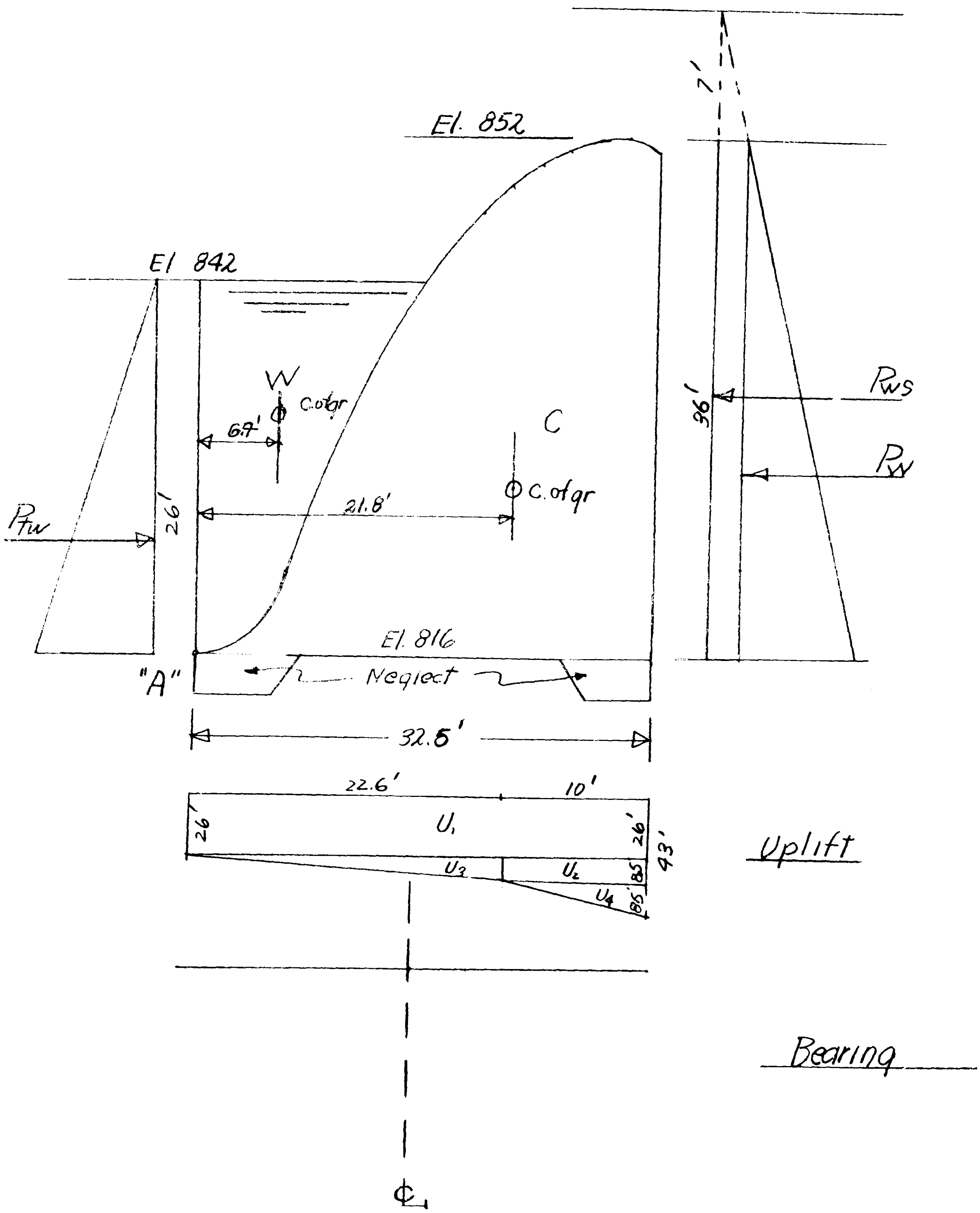
WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Subject: Birch Hill - Spillway - Max. Section (36') - 7' surcharge

Computation: Stability

Computed by: R.H.M. Checked by: Date:



WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page 40

Subject Birch Hill – Spillway – Max Section (36') – 7' surcharge

Computation Stability

Computed by R.H.M. Checked by ______ Date ______

Forces Acting		↓	↑	→	←	Arm	Moments about "A" ↻	Moments about "A" ↺
C	673.74 × 150	101,060				21.9	2,213,200	
W	279.90 × 62.5	17490				6.4	111,900	
U_1	26 × 62.5 × 32.6		52980			16.3		863600
U_2	8.5 × 62.5 × 10		5310			27.6		146,600
U_3	8.5 × 62.5 × ½ × 22.6		6000			15.1		90,600
U_4	8.5 × 62.5 × ½ × 10		2660			29.3		77,900
P_{WS}	7 × 62.5 × 36				15750	18.0		283,500
P_W	$36^2 \times \frac{1}{2} \times 62.5$				40500	12.0		486000
P_{TW}	$26^2 \times \frac{1}{2} \times 62.5$			21100		8.67	183000	
		118550	66950	21100	56250		2,508,100	1,948,200
		ΣV = 51600 #↓		ΣH = 35150 #←			ΣM = 559,900 '# ↻	

$$\frac{\Sigma M}{\Sigma V} = \frac{559{,}900}{51600} = 10.85' \qquad \frac{1}{3}\text{ Base} = \frac{32.6}{3} = 10.86' ; \; e = 16.25 - 10.85 = 5.4$$

$$B.P. = \frac{51600}{32.5}\left(1 \pm \frac{6 \times 5.4}{32.5}\right) = 3180 \text{ \#/}\square' \text{ to } 0$$

Subject: BIRCH HILL
Computation: Access road bridge
Computed by: RSM Checked by: Date: 1-2-40

As this bridge is located alongside a railroad fill with a drain between the roadway and railroad, it is possible for earth fill behind the abutment to be fully saturated near the drain and the abutment is designed for this condition, with an added surcharge for live loading.

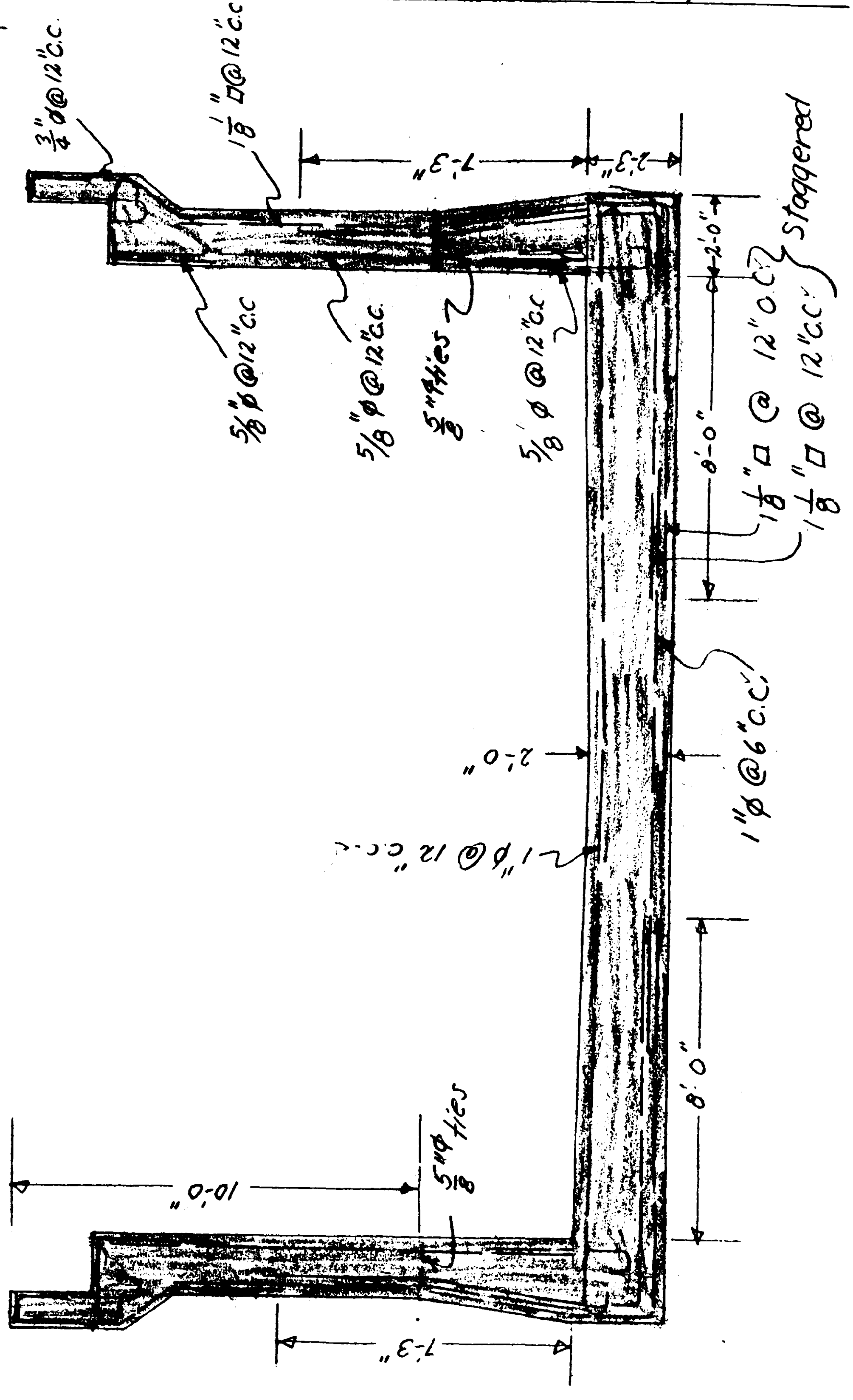
The upstream wingwall is designed for full saturation, but with no surcharge.

The downstream wingwall is designed for dry earth, as it is felt that the line of saturation will be low at this section due to relief through weepholes and through the side of the fill.

rough surcharge storage to produce ... th

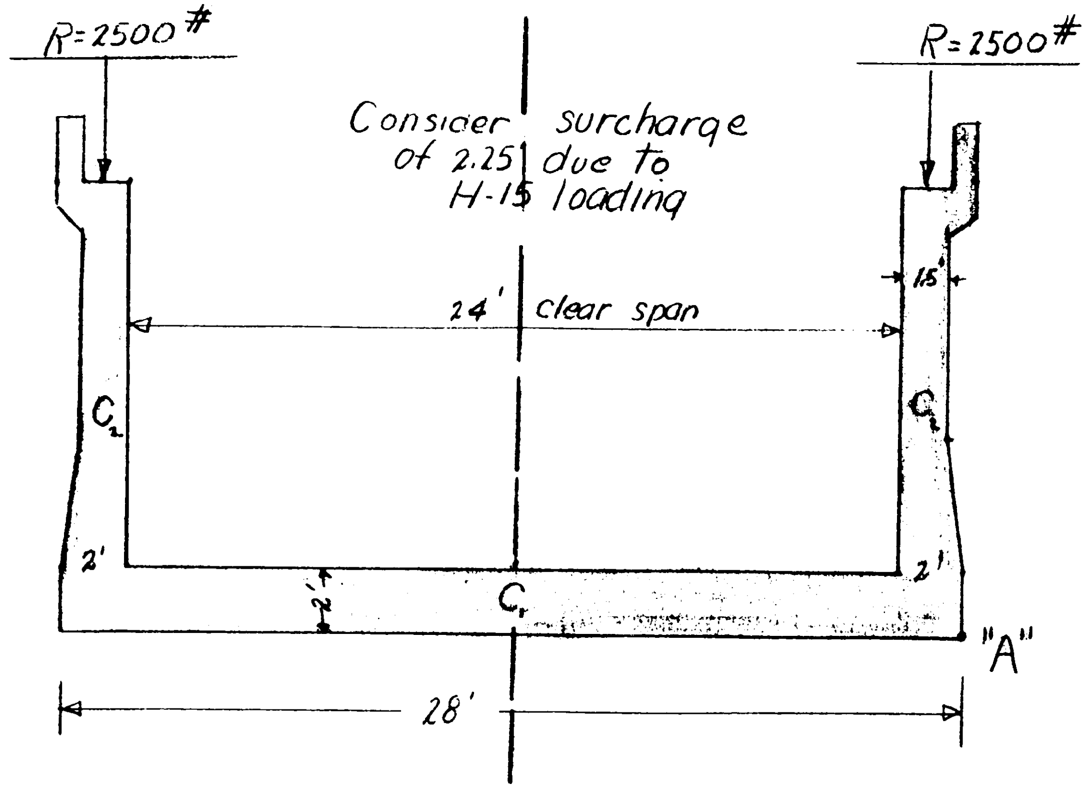
Subject: Birch Hill
Computation: Access Road bridge
Computed by: RHM Checked by: R.W. Date: 12-8-39

ABUTMENT



Subject Birch Hill –
Computation Access road bridge
Computed by R.H.M Checked by RW Date 12/8/39

ABUTMENT



Case I No uplift (Equal earth pressures) (a) Neglect L.L

C_1 : 2×28×150 = 8400
C_2 : 2×1.5×12×150 = 5400
R : 2×2500 = 5000
ΣV = 18800 # ↓

$$B.P = \frac{18800}{28} = 670 \ \#/\square'$$

(b) Including L.L (heavy wheel on reaction – other on span)

C_1	2×28×150	= 8440 × 14	= 118160	'#
C_2	2×1.5×12×150	= 5400 × 14	= 75600	"
R	2×2500	= 5000 × 14	= 70000	"
R_2	2540	= 2540 × 26.75	= 67950	"
R_2'	310	= 310 × 1.25	= 390	"
	ΣV =	21690 # ↓	ΣM = 332,100	'#

$$\frac{\Sigma M}{\Sigma V} = \frac{332100}{21690} = 15.31' \qquad e = 15.31 - 14.00 = 1.31'$$

Subject Birch Hill
Computation Access road bridge
Computed by R. H. M. Checked by R.W. Date 12-8-39

(a) At center of base slab (Bottom steel)

Case I(a) governs

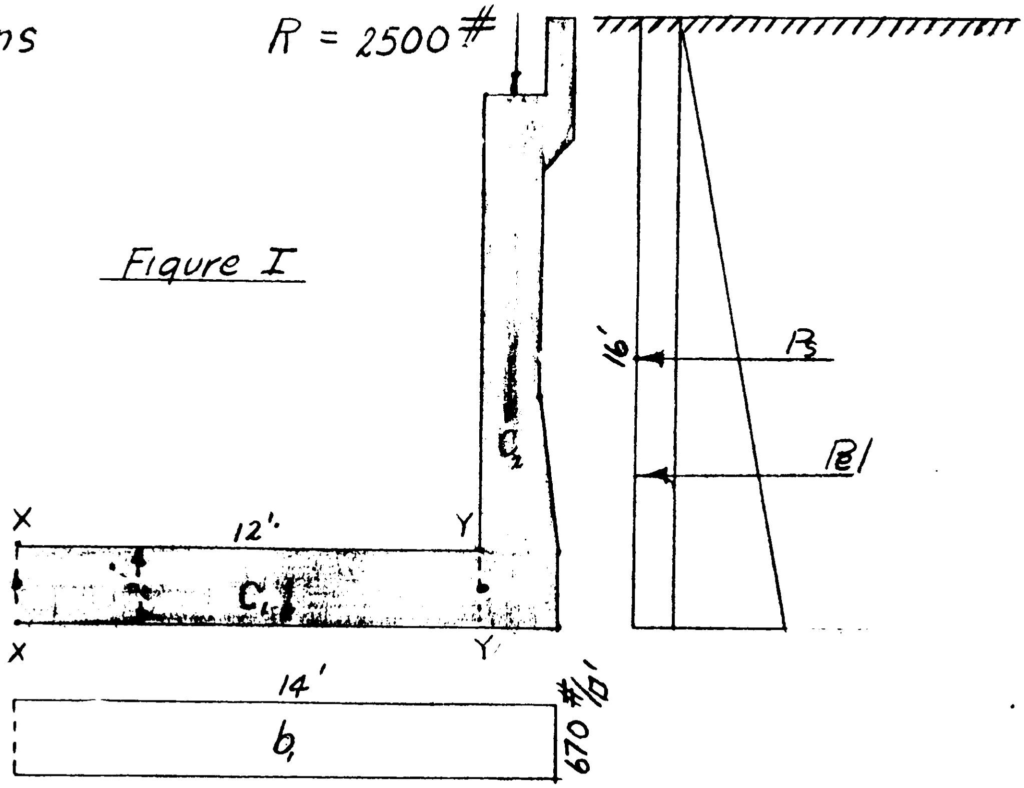


Figure I

At section x·····x

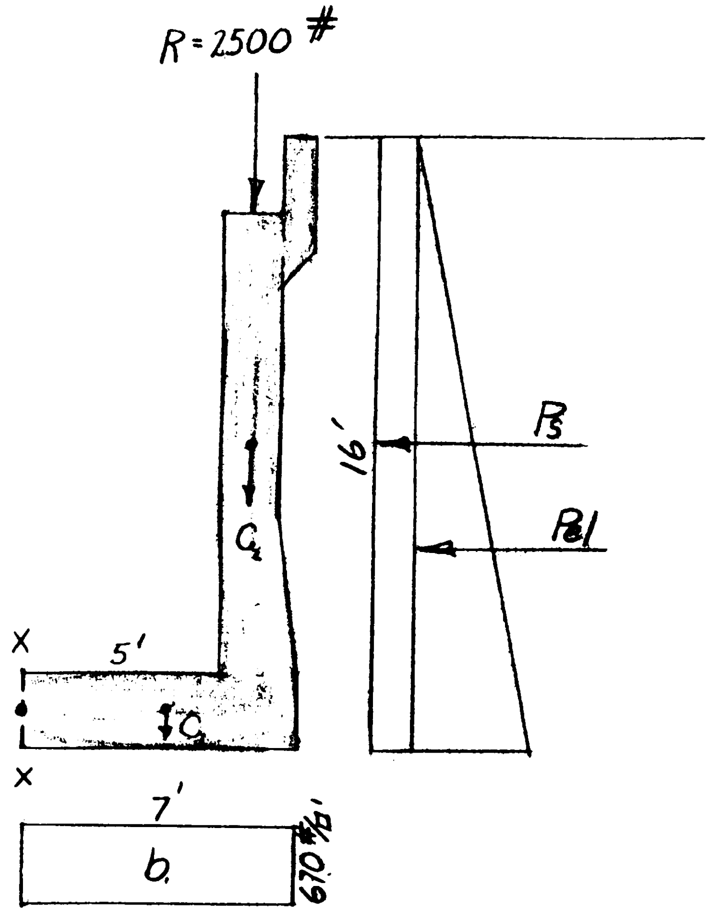
↓ R :	2500·	= 2500·× 12.75·	= 31880·	1#↻
↓ C_2 :	12×1.5×150·	= 2700·× 12.75·	= 34430·	"
↓ C_1 :	14×2.0×150	= 4200·× 7.00·	= 29400·	"
↑ b_1 :	670·×14	= 9380·× 7.00·	= 65660·	1#↺
← Ps :	2.25×80×16	= 2880·× 7.00·	= 20160·	"
← Pel :	$16^2\times\frac{1}{2}\times 80$	= 10,240·× 4.33·	= 44340·	"

ΣV = 20 #↓ ΣM = 34450· 1#↺

approx. $d = \sqrt{\frac{34450}{123}} = \sqrt{280} = 16.8" + 4.5" = 21.3"$ needed OK.
24" provided

$$A_s = \frac{12 \times 34450}{18000 \times .884 \times 19.5} = \frac{413400}{310,000} = 1.33 \square''$$

ubject Birch Hill
mputation Access road bridge
omputed by RHM Checked by RW Date 12-8-39



At Section x-----x

↓	R :	2500 =	2500 × 5.75 =	14380'# ↻
↓	C_2 :	12×1.5×150 =	2700 × 5.75 =	15530 "
↓	C_1 :	7×2.0×150 =	2100 × 3.50 =	7350 "
↑	b :	670 × 7 =	4690 × 3.50 =	16420'# ↺
←	P_s :	2.25×80×16 =	2880 × 7.00 =	20160 "
←	Pel :	$16^2 \times \frac{1}{2} \times 80$ =	10240 × 4.33 =	44340 "

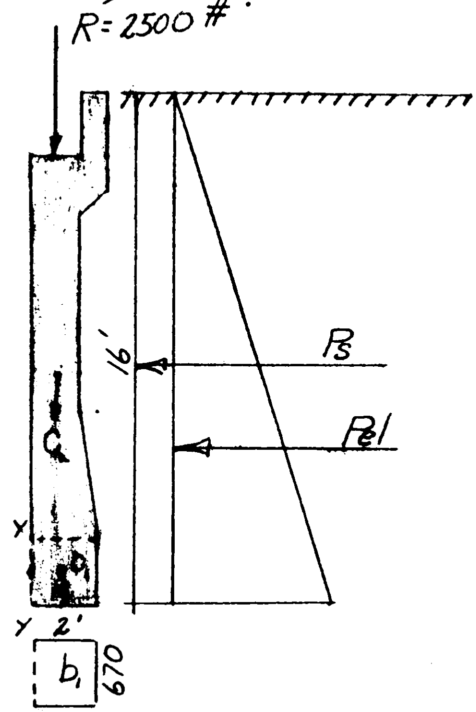
$\Sigma V = 2610\#\downarrow$ $\qquad \Sigma M = 43660'\#$ ↺

$$A_s = \frac{43660 \times 12}{18000 \times .884 \times 21} = \frac{523900}{333900} = 1.57\square''$$

Subject Birch Hill
Computation Access road bridge
Computed by RHM Checked by RW Date 12-8-39

(b) At inside face of stem (Bottom steel)

At section Y---Y



↓ R	: 2500	=	2500 × .75	=	1880	'# ↺
↓ C_2	: 12×1.5×150	=	2700 × .75	=	2020	"
↓ C_1	: 2×2.0×150	=	600 × 1.00	=	600	"
↑ b_1	: 2 ×670	=	1340 × 1.00	=	1340	'# ↻
← Ps	: 2.25×80×16	=	2880 × 7.00	=	20160	"
← Pel	: $16^2 \times \frac{1}{2} \times 80$	=	10240 × 4.33	=	44340	"
	ΣV = 4,460 #↓				61340	'#

Trial $d = 27 - 4.5 = 22.5''$ $J = .884$

" $A_s = \dfrac{12 \times 61340}{18000 \times .884 \times 22.5} = \dfrac{736080}{358000} = 2.06 \square''$

WAR DEPARTMENT

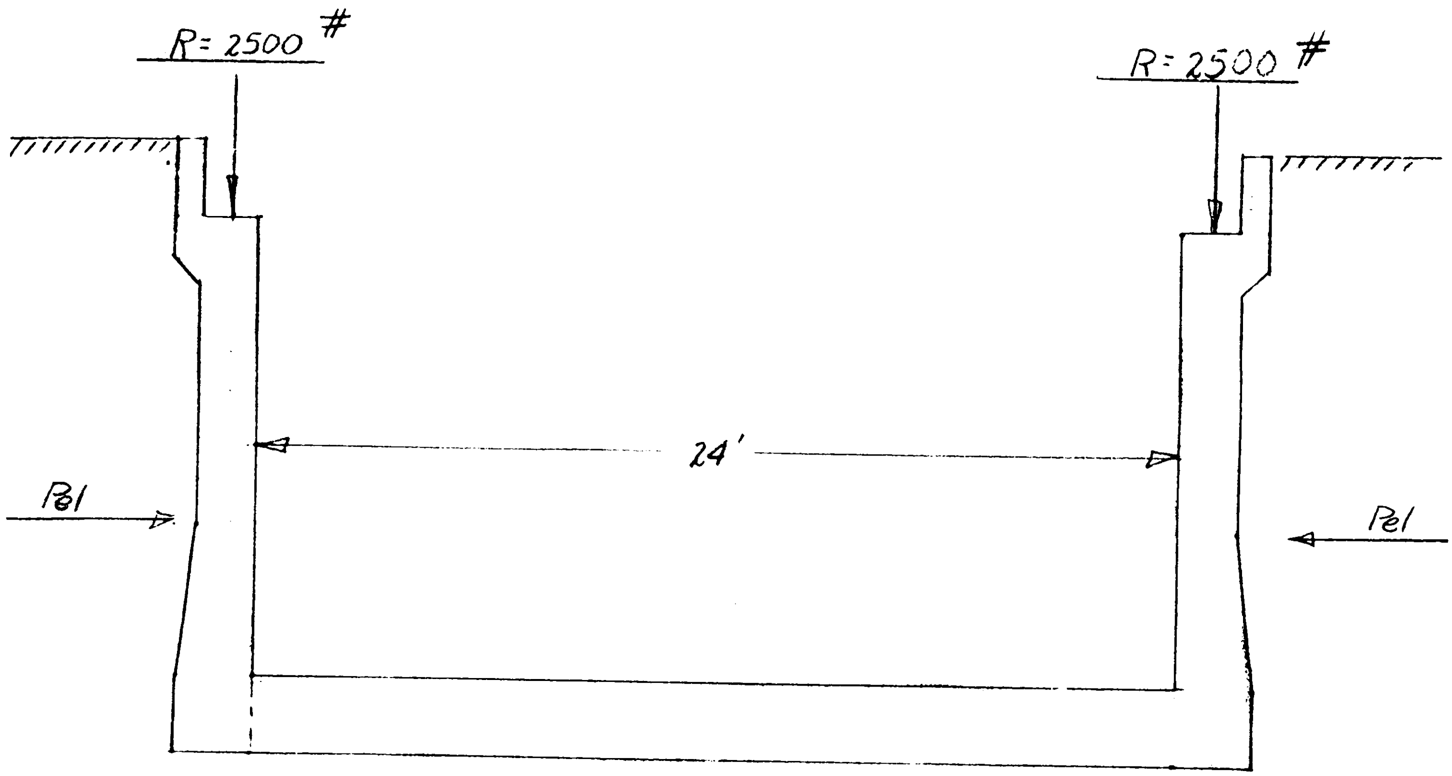
U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

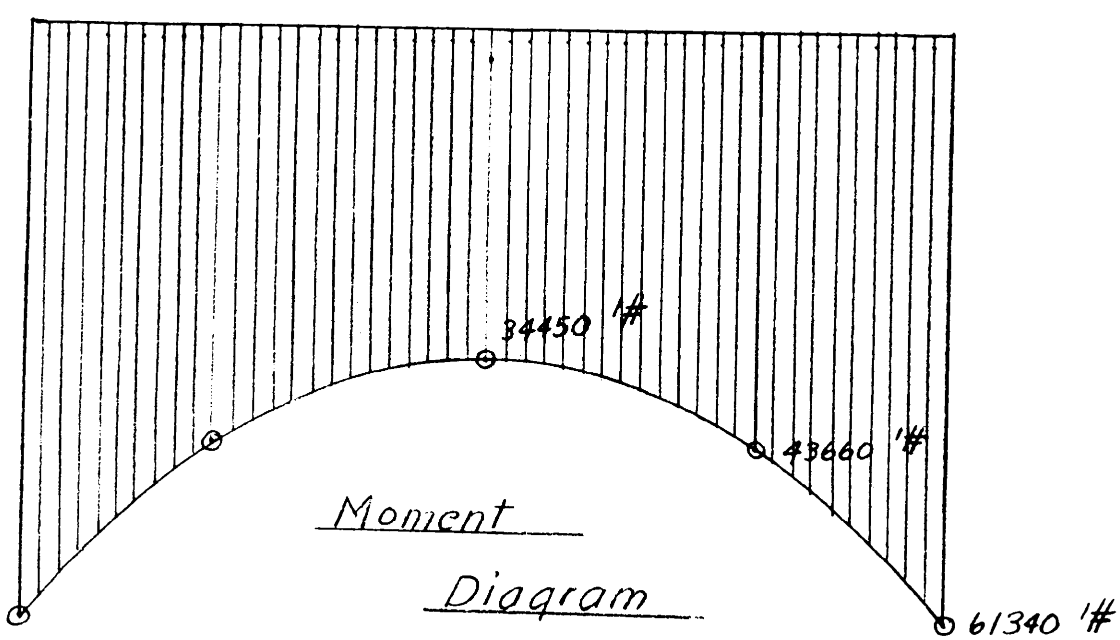
Page 47

ubject Birch Hill

mputation Access road bridge

omputed by RHM Checked by Date 12-8-39





WAR DEPARTMENT

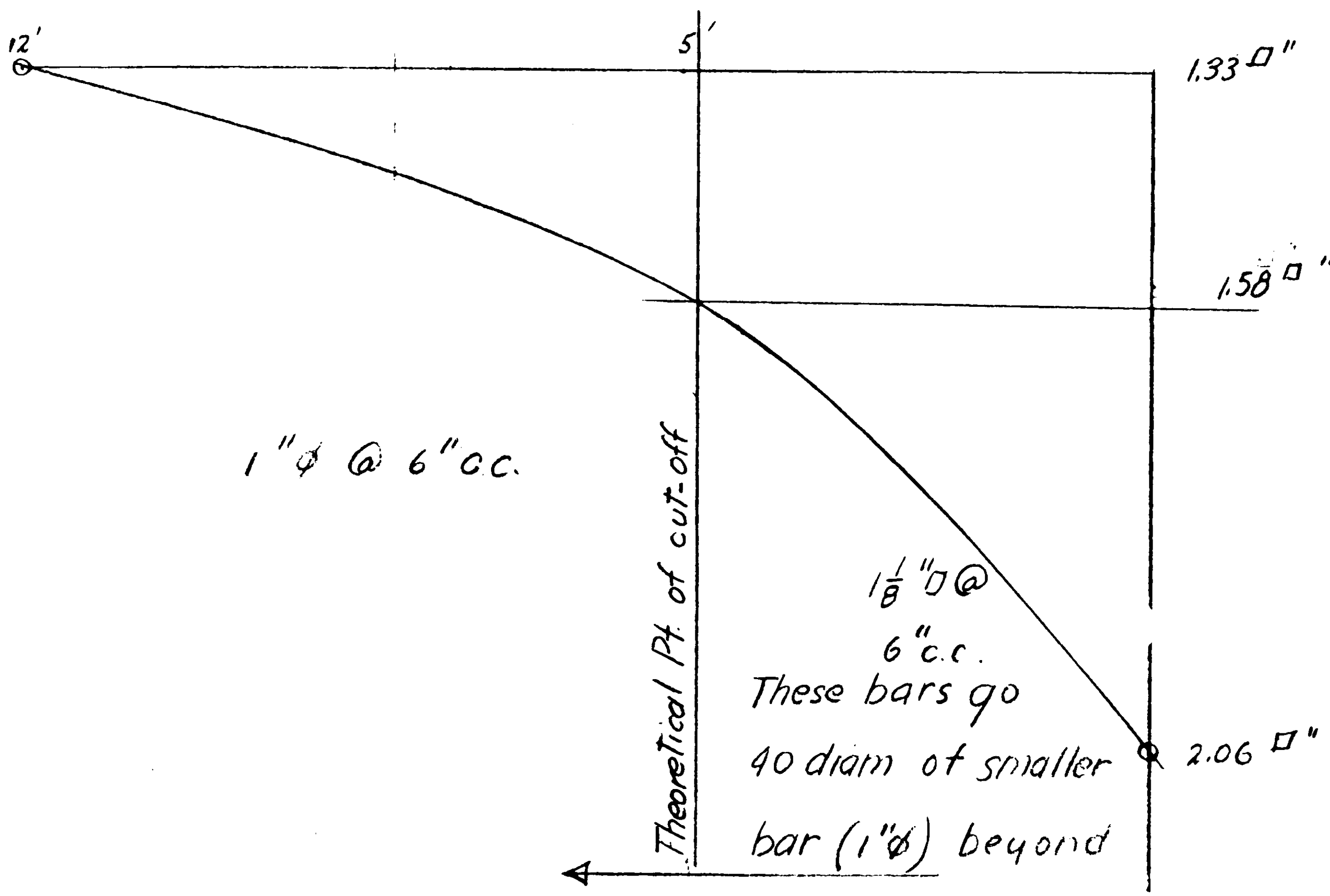
U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page 48

ubject Birch Hill

mputation Access road bridge

omputed by PHM Checked by R.W. Date 12-8-39



Subject: Birch Hill
Computation: Access road bridge
Computed by: RHM Checked by: RW Date: 12-8-39

(a) At center of base slab (Top steel)

Case I (b) governs (see figure I)

↓ R_1 :	2500	=	2500	×	12.75	= 31880 '#
↓ R_2	310	=	310	×	12.75	= 3950 "
↓ C_2 :	12×1.5×150	=	2700	×	12.75	= 34430 "
↓ C_1 :	14×20×150	=	4200	×	7.00	= 29400 "
↑ b_1 :	560 × 14	=	7840	×	7.00	= 54880 '#
↑ b_2 :	215×½×14	=	1510	×	4.67	= 7050 "
← Pel :	16² × ½ × 35	=	4480	×	4.33	= 19400 "
ΣV = 360 # ↓					ΣM	= 18330 '#

(b) At inside face of stem

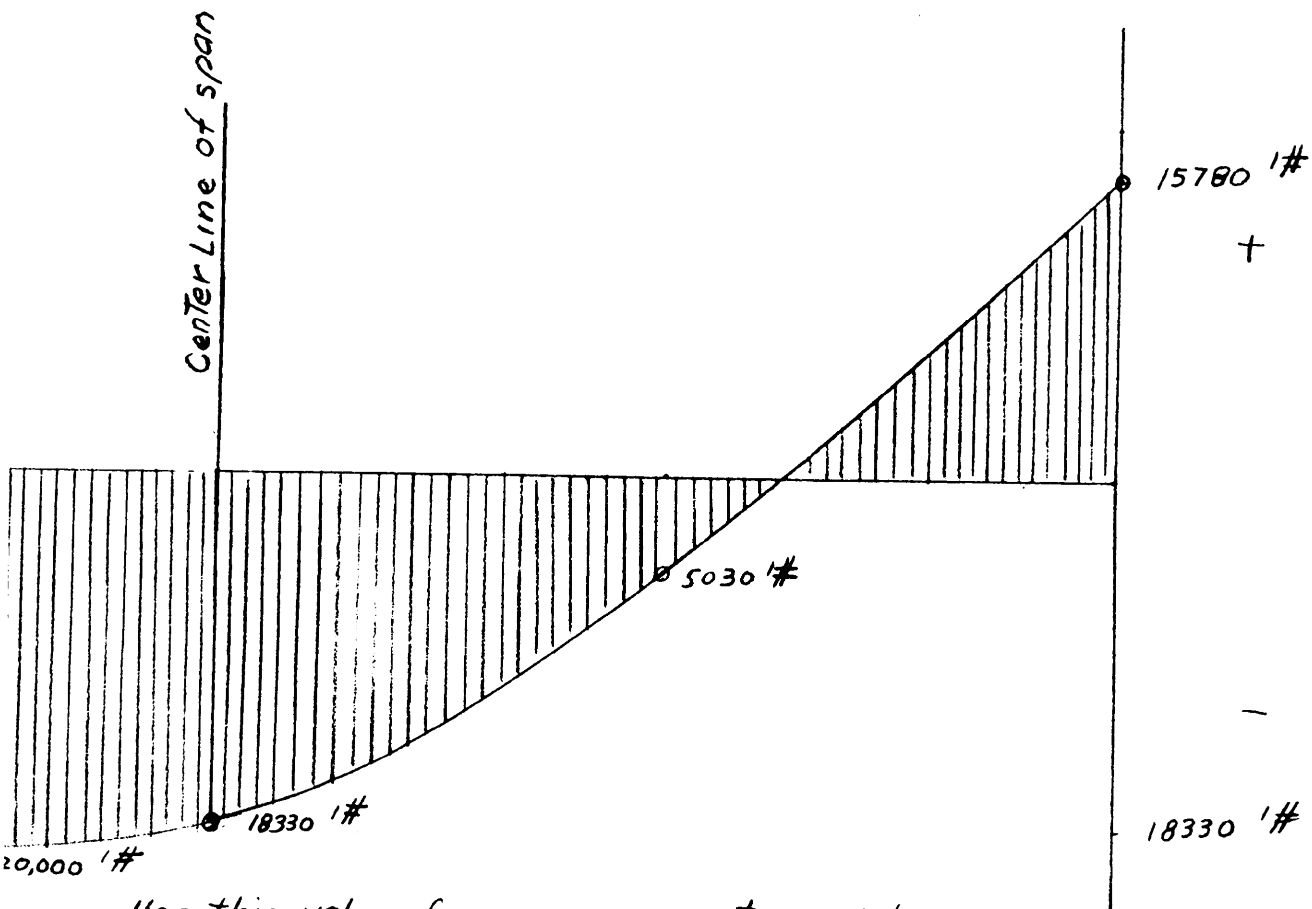
Case I (b) governs

↓ R_1 :	2500	=	2500	×	.75	= 1880 '#
↓ R_2 :	310	=	310	×	.75	= 230 "
↓ C_2 :	2700	=	2700	×	.75	= 2030 "
↓ C_1	2×2×150	=	600	×	1.00	= 600 "
↑ b_1 :	560×2	=	1120	×	1.00	= 1120 '#
← Pel :	4480	=	4480	×	4.33	= 19400 "
			ΣV = 4990 # ↓			ΣM = 15780 '#

(c) At Point 5' from inside face of stem

↓ R_1 :	2500	=	2500	×	5.75	= 14380 '#
↓ R_2 :	310	=	310	×	5.75	= 1780 "
↓ C_2 :	2700	=	2700	×	5.75	= 15530 "
↓ C_1 :	7×2×150	=	2100	×	3.50	= 7350 "
↑ b_1 :	560×7	=	3920	×	3.50	= 13720 '#
↑ b_2 :	7×108×½	=	380	×	2.33	= 890 "
← Pel :	4480	=	4480	×	4.33	= 19400 "
			ΣV = 3330 # ↓			5030 '#

ject Birch Hill
omputation Access road bridge
omputed by RHM Checked by R.W. Date 12-8-39



Use this value for max. moment in determining top steel in base

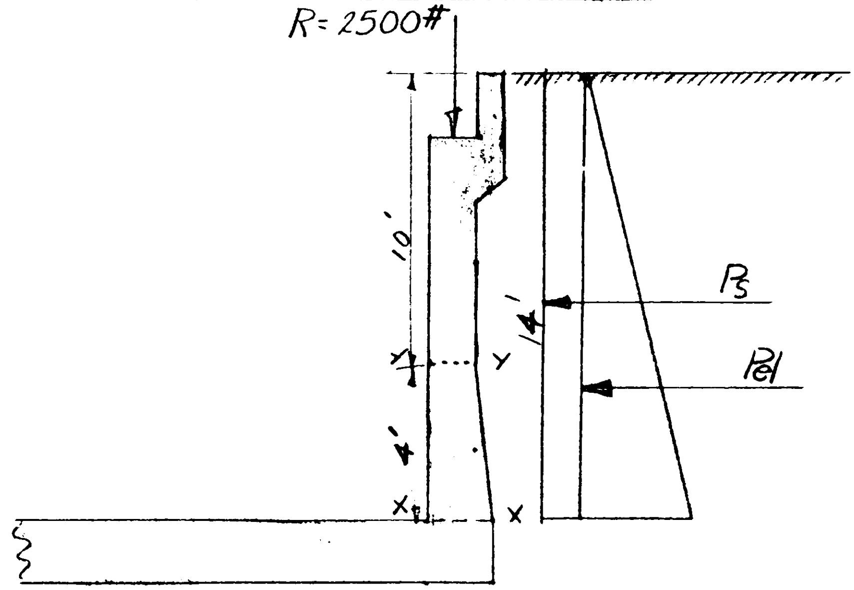
$$A_s = \frac{20000 \times 12}{18000 \times .884 \times 20.5} = \frac{240,000}{326,000} = .74 \ \square''$$

Use 1"ϕ @ 12" c.c.

ject Birch Hill
mputation Access road bridge
mputed by RHM Checked by RW Date 12-8-39

DESIGN OF STEM

Stem assumed to act as a cantilever



Moment at section x···x $n=12$; $f_s=18000$ $f_c=800$; $j=.884$

$\leftarrow P_s : 2.25 \times 80 \times 14 = 2520 \times 7.00 = 17640$ '#
$\leftarrow P_{el} : 14^2 \times \frac{1}{2} \times 80 = 7840 \times 4.67 = 36610$ "

$\Sigma H = 10360\#\leftarrow$ $\Sigma M = 54{,}250$ '#

$$d=\sqrt{\frac{54250}{123}} = \sqrt{441} = 21 + 3.5 = 24.5''$$

$$v = \frac{10360}{12 \times .884 \times 20.5} = \frac{10360}{217} = 48\#/\square'' \text{ OK.}$$

Moment at section Y···Y

$\leftarrow P_s : 2.25 \times 80 \times 10 = 1800 \times 5.00 = 9000$ '#
$\leftarrow P_{el} : 10^2 \times \frac{1}{2} \times 80 = 4000 \times 3.33 = 13320$ "

$\Sigma H = 5800\# \leftarrow$ 22320 '#

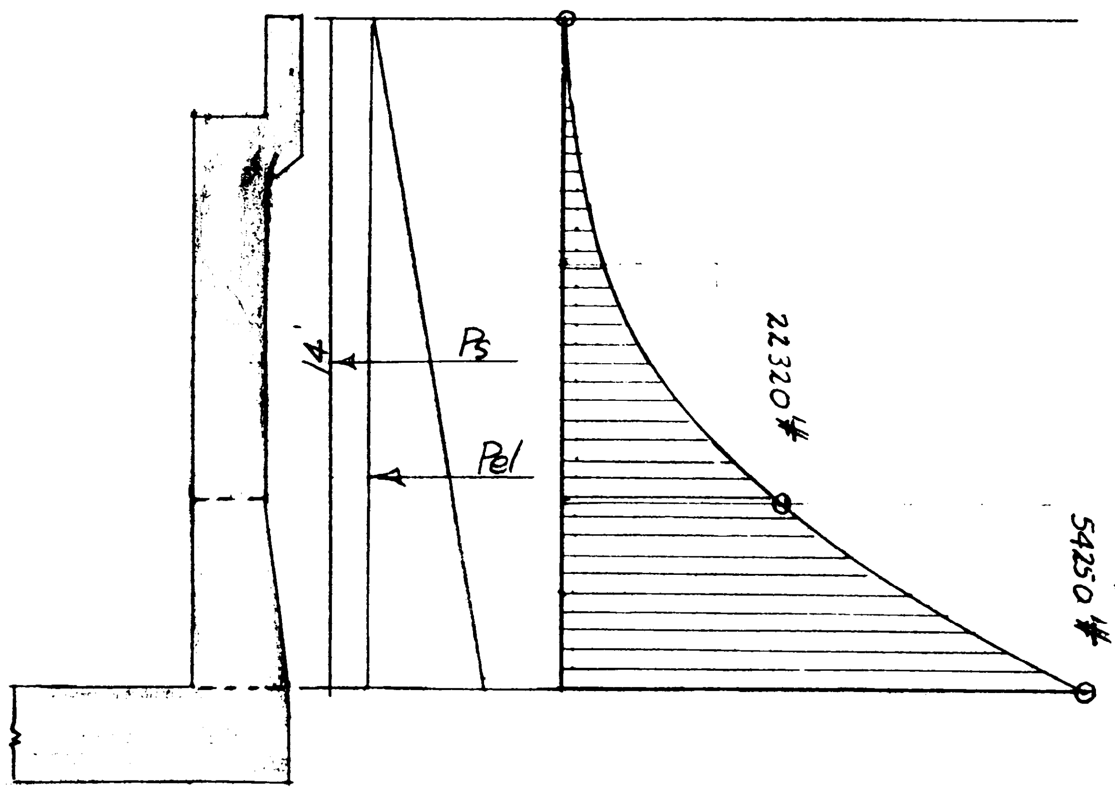
$$d = \sqrt{\frac{22320}{123}} = \sqrt{181} = 13.5 + 3.5 = 17'' \text{ O.K. } 18'' \text{ provided}$$

$$v = \frac{5800}{12 \times .884 \times 14.5} = \frac{5800}{154} = 38\#/\square''$$

Project Birch Hill
Computation Access road bridge
Computed by RHM Checked by R.W. Date 12-8-39

Earth Pressure Diagram

Bending Moment Diagram



WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

ject Birch Hill

omputation Access road bridge

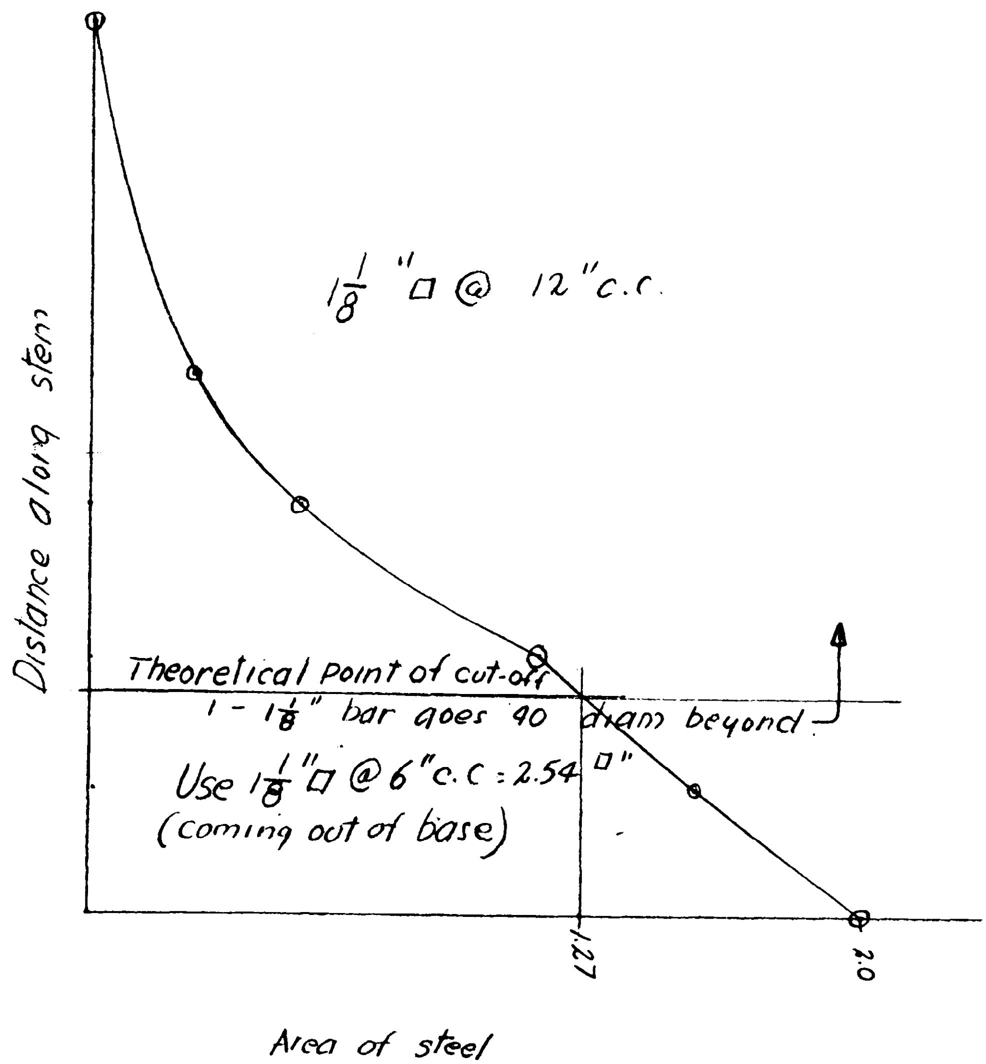
omputed by RHM Checked by R.W. Date 12-8-39

At section X--X

$$A_s = \frac{12 \times 54250}{18000 \times .884 \times 20.5} = \frac{651000}{325950} = 2.00\ \square''$$

At Section Y--Y

$$A_s = \frac{12 \times 22320}{18000 \times .884 \times 14.5} = \frac{267840}{231000} = 1.16\ \square''$$

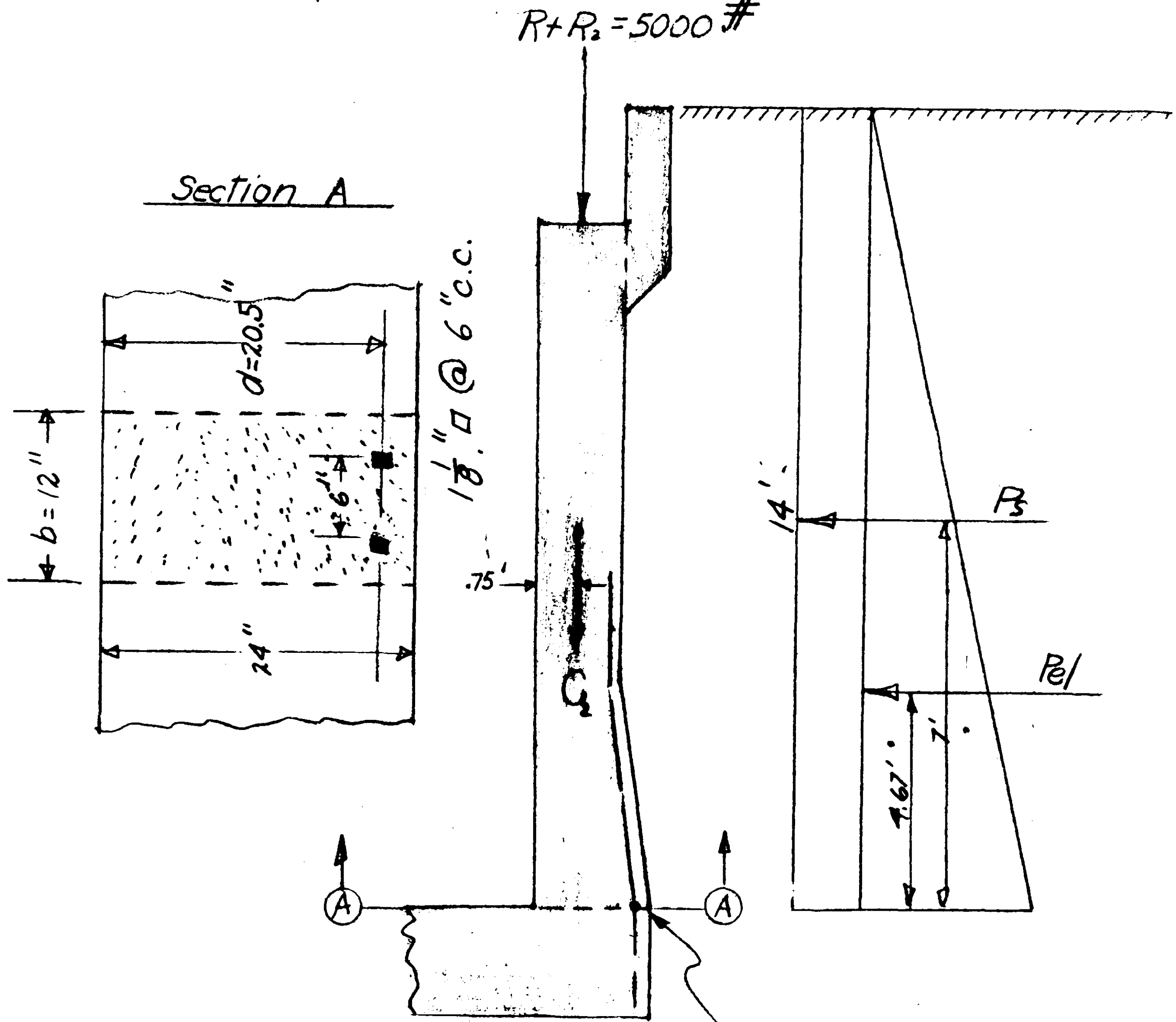


WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

[Proj]ect: Birch Hill
[Com]putation: Access road bridge
[Com]puted by: RHM Checked by: R.W. Date: 12-8-39

$n = 12$, $A_s = 2.54 \square''$ ($1\frac{1}{8}'' \square$ bars @ 6" c.c.)
Trial



Moments taken about center of steel (in inch lbs)

← P_s:	$2.25 \times 80 \times 14$	$= 2520 \times 84$	$= 211680$ "#	↶
← Pel:	$14^2 \times \frac{1}{2} \times 80$	$= 7840 \times 56.04$	$= 439350$ "	"
↓ C_2:	2700	$= 2700 \times 11.50$	$= 31100$ "	"
↓ $R+R_2$:	5000	$= 5000 \times 11.50$	$= 57500$ "	"

$N = 7700\#\downarrow$ $\Sigma M = 739600$ "# ↶

$$e' = \frac{739600}{7700} = 96.1''$$

$$p = \frac{A_s}{bd} = \frac{2.54}{12 \times 20.5} = \frac{2.54}{246} = .0103$$

To find k

$$k^3 + 3k^2\left(\frac{e'}{d} - 1\right) - 6np(1-k)\frac{e'}{d} = 0$$

$$k^3 + 3k^2\left(\frac{96.1}{20.5} - 1\right) - 6 \times 12 \times .0103(1-k)\left(\frac{96.1}{20.5}\right) = 0$$

$$k^3 + 11.07k^2 + 3.478k - 3.478 = 0$$

try $k = .419$

1	+ 11.0700	+ 3.478	− 3.478
	+ .419	+ 4.814	+ 3.474
	11.489	8.292	.000

Use .419

$$j = 1 - \frac{k}{3} = 1 - \frac{.419}{3} = 1 - .140 = .860$$

$$f_c = \frac{2Ne'}{kj\,b \cdot d^2} = \frac{2 \times 7700 \times 96.1}{.419 \times .860 \times 12 \times 20.5^2} = \frac{1{,}479{,}200}{1820} = 810 \#/\square''$$

O.K.

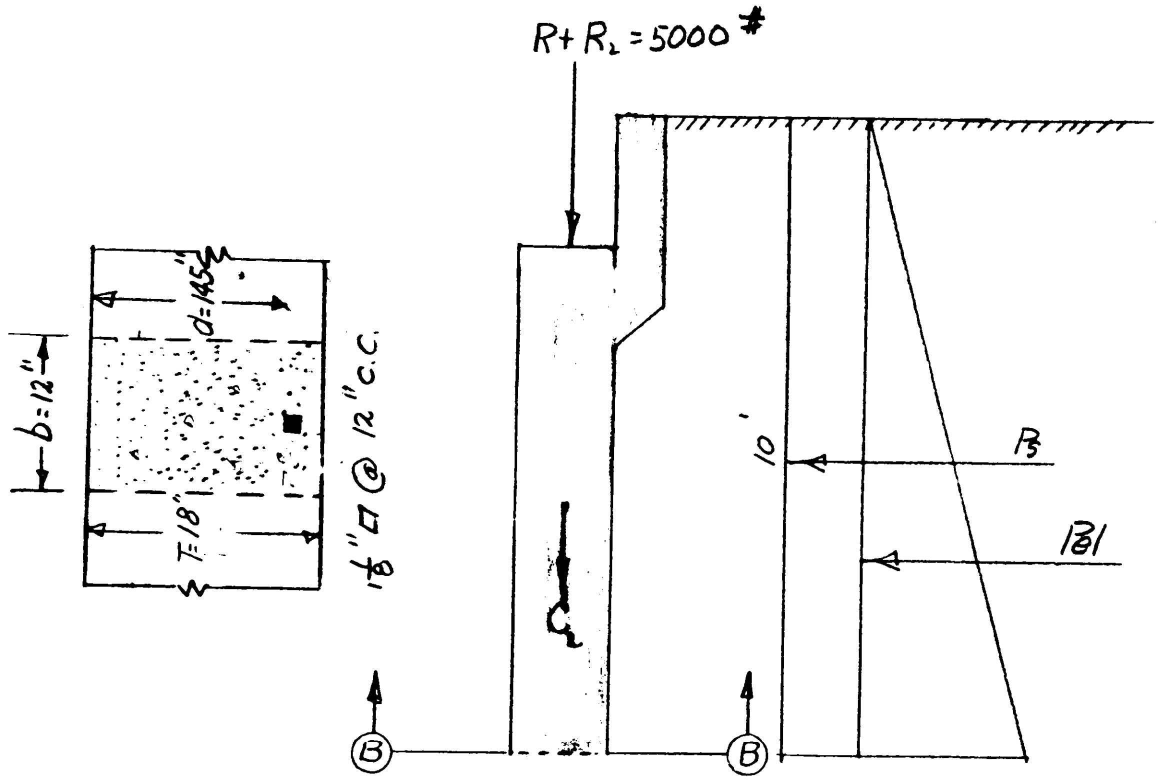
$$f_s = n \cdot f_c\left(\frac{1-k}{k}\right) = 9720 \times \left(\frac{1-.419}{.419}\right) = 1.40 \times 9720 = 13600 \#/\square''$$

These stresses accompany severest live load conditions.

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Subject: Birch Hill
Computation: Access road bridge
Computed by: RHM Checked by: RW Date: 12-8-39



Trial A_s.

P_s : $2.25' \times 80' \times 10' = 1800' \times 5.00 = 9000'^{\#}$
P_{el} : $10^2 \times \frac{1}{2} \times 80' = 4000' \times 3.33 = 13320'^{\#}$

$\Sigma M = 22320'^{\#}$

Use $j = .884$

$$A_s = \frac{M}{f_s \cdot j d} = \frac{12 \times 22320}{18000 \times .884 \times 14.5} = \frac{267840}{231{,}000} = 1.16 \square''$$

Using $1\frac{1}{8}$" □ @ 12" c.c. we get an A_s of 1.27 □".
Using this A_s the section will be investigated by the method of Bending and Direct stress to ascertain that the allowable stresses are not exceeded.

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

bject: Birch Hill

omputation: Access road bridge

omputed by: RHM Checked by: R.W. Date: 12-8-39

Live Load giving severest conditions

Moments taken about center of steel (in inch lbs)

$\leftarrow P_s : 2.25 \times 80 \times 10 = 1800 \times 60 = 108000$ "#

$\leftarrow P_{el} : 10.^2 \times \frac{1}{2} \times 80 = 4000 \times 39.96 = 159840$ "

$\downarrow C_2 : 1.5 \times 8 \times 150 = 1800 \times 5.50 = 9900$ "

$\downarrow R+R_2 : = 5000 \times 5.50 = 27500$ "

$\Sigma H = 5800\#\leftarrow \quad N = 6800\#\downarrow \quad 305240$ "#

$$e' = \frac{305240}{6800} = 44.9''$$

$$p = \frac{A_s}{bd} = \frac{1.27}{12 \times 14.5} = \frac{1.27}{174} = .0073$$

To find k

$$k^3 + 3k^2\left(\frac{44.9}{14.5} - 1\right) - 6 \times 12 \times .0073(1-k)\frac{44.9}{14.5} = 0$$

$$k^3 + 6.30k^2 + 1.63k - 1.63 = 0$$

try k = .386

1	+ 6.3000	+1.6300	− 1.630	
	+ .386	+2.585	+1.630	
	6.686	4.215	+0.000	

Use $k = .386$

$$J = 1 - \frac{k}{3} = 1 - \frac{.386}{3} = 1 - .129 = .871$$

$$f_c = \frac{2Ne'}{k\,j\,bd^2} = \frac{610480}{.386 \times .871 \times 12 \times 14.5^2} = \frac{610480}{848} = \underline{720\ \#/\square''}$$

$$f_s = n f_c\left(\frac{1-k}{k}\right) = 12 \times 720\left(\frac{1-.386}{.386}\right) = 8740 \times 1.59$$

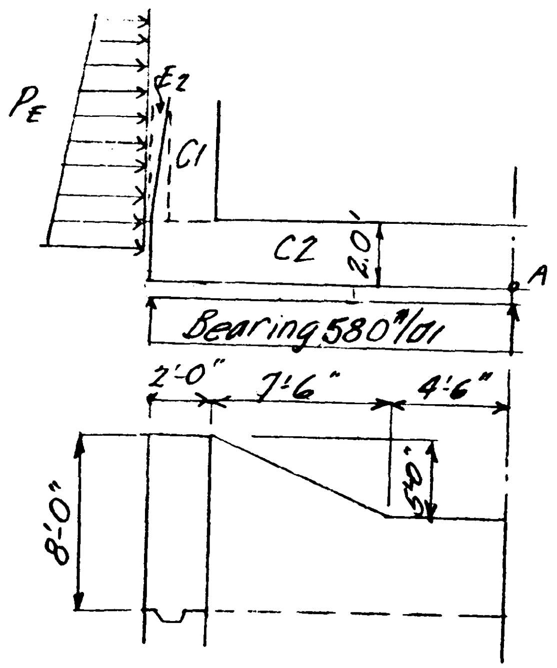
14000 #/□"

13752 ok

Subject BIRCH HILL
Computation Access road bridge
Computed by RSM Checked by Date 12-14-39

WINGWALLS - UPSTREAM

DESIGN OF BASE



$\rightarrow M = \frac{14.8^3}{6} \times 80 =$ 43200'#

$\downarrow V = 14 \times 2 \times 150 =$ 4200#
$13.8 \times 1.5 \times 150 =$ 3100#
$13.8 \times .5 \times 125 =$ 860
8160#

Bearing = Σ Bearing + uplift
Assume uniform distribution
Loading decreases upstream, due to support from existing wall.

$b = 8160/14 = 580$ #/□'

At 2'6" from face of stem. V

		V	M ↻	M ↺
$\rightarrow P_E$			43200'#	
$\downarrow C_1$		3100 × 3.3 =		10200
E_2		860 × 4.3 =		3700
C_2	150 × 4.5 × 2 =	1350 × 2.3 =		3100
↑ B	580 × 4.5 =	2620 × 2.3 =	6000	
	V =	#	49,200 '#	17,000 '#

ΣM 32,200'#

$A_s = 32{,}200 \times 12 / 18000 \times .867 \times 19.5 = 1.27$ □"
$v = 2690 / 12 \times .867 \times 19.5 = 13$ #/□"

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page 59

bject BIRCH HILL

mputation Access road bridge

mputed by RSM Checked by ______ Date 12-14-39

WINGWALLS - UPSTREAM

2. At face of stem

		V		M→	M←
→ P_E				43200	
↓ C1		3100	×.7		2170
↓ E2		860	× 1.8		1550
↓ C2	150×2×2	600	× 1.0		600
↑ B	910×2	1820	× 1.0	1820	
		2740		45020	4,320
				Σ 40700 '#	

$A_s = 40700 \times 12 / 18000 \times .867 \times 19.5 = 1.60^{\square''}$

$v = 2740 / 12 \times .867 \times 19.5 = 14^{\#}/\square''$

3. At 7'-6" from face of stem (consider full section - 8' wide)

		V		M→	M←
→ P_E				346,000 '#	
↓ C1		24800	× 8.3		206,000 '#
↓ C2	9.5×2×3×150 =	8550	×4.8		41,000
	2×2×5×150 =	3000	×8.5		26,000
	$\frac{5 \times 7.5 \times 2}{2} \times 150$ =	5620	×5.0		28,000
↓ E2		6880	× 9.3		64,000
↑ B	910×9.5×3	25900	×4.8	124,000	
	2×5×910	9100	×8.5	77,000	
	$\frac{5 \times 7.5}{2} \times 910$	17000	×5.0	85,000	
		3150		632,000	365,000
				Σ 267,000 '#	

$A_s = 267000 \times 12 / 18000 \times .867 \times 19.5 = 10.5^{\square''}$, distributed over 3' width

ject: BIRCH HILL
mputation: Access road bridge
mputed by: RSM Checked by: Date: 12-15-39

WINGWALLS - UPSTREAM

DESIGN OF BASE

2. At face of stem - use steel as bent in from stem

		V		M→	M←
→ P_E =				43200 '#	
↓ C1 =		3100#	× .7		2170
↓ E2 =		860	× 1.8		1550
↓ C2 =	150×2×2	600	× 1.0		600
↑ B =	580×2	1160	× 1.0	1160	
		3400#		44360 '#	4320
				Σ 40020 '#	

A_s = 40020 × 12/18000 × .867 × 19.5 = 1.57 □"

v = 3400/12 × .867 × 19.5 = 17#/□"

3. At 7.5' from face of stem

		V		M→	M←
→ P_E				43200 '#	
↓ C1		3100#	× 8.2		25400
↓ E2		860	× 9.3		8000
↓ C2 =	150×9.5×2	2850	× 4.8		13600
↑ B =	580×9.5	5500	× 4.8	26400	
		1310		69600	47000
				Σ 22600 '#	

A_s = 22600 × 12/18000 × .867 × 19.5 = .89 □"

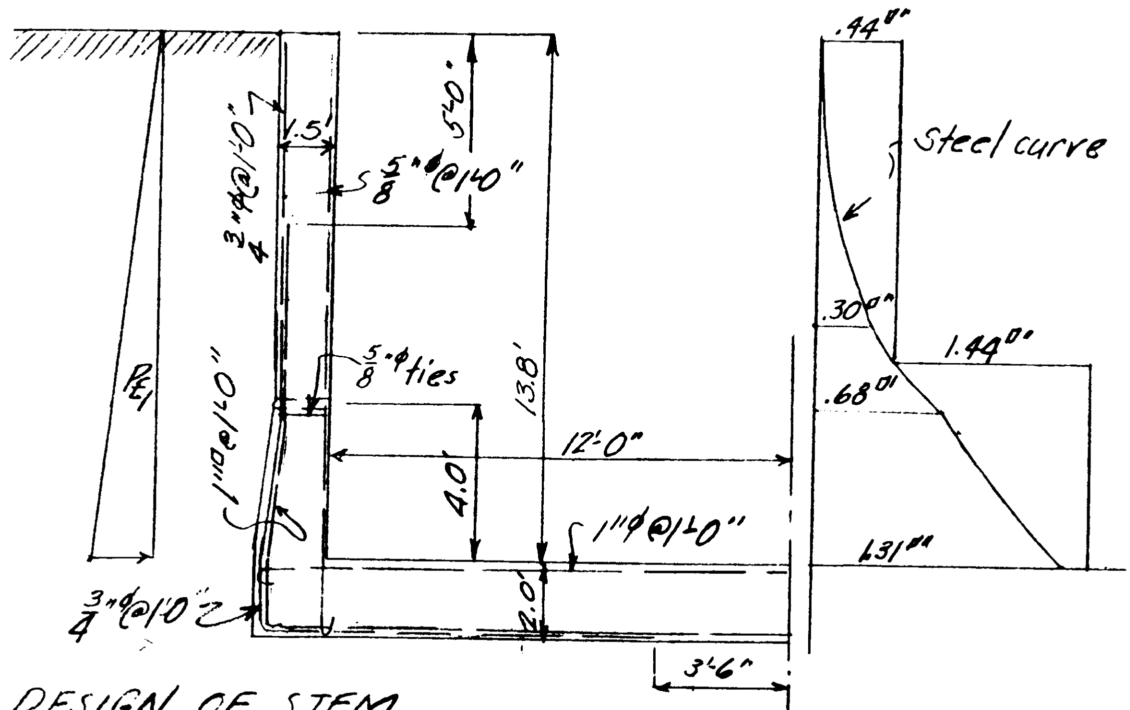
4. At ℄ of slab

		V		M→	M←
→ P_E				43200 '#	
↓ C1		3100	× 12.7		39300
↓ E2		860	× 13.8		11900
↓ C2 =	14×2×150	4200	× 7.0		29400
↑ B	580×14	8130	× 7.0	57000	
				100,200	80600
				ΣM = 19,600 '#	

A_s = 19600 × 12/18000 × .867 × 19.5 = .78 □"

Project BIRCH HILL
Computation Access road bridge
Computed by RSM Checked by Date 12-12-39

WINGWALLS - UPSTREAM



DESIGN OF STEM

1. At base

P_{E1} $\quad 80 \times \overline{13.8}^2/2 = \quad$ V $7640^{\#} \times 4.6 = \quad$ M $35{,}100'^{\#}$

$A_s \quad 35100 \times 12 / 18000 \times .867 \times 20.5 = 1.31^{\square''}/ft.$

$v \quad 7640/12 \times .867 \times 20.5 = 36^{\#}/_{\square''}$

2. At haunch

$P_{E1} = 80 \times \overline{9.8}^2/2 = \quad$ V $3840^{\#} \times 3.3 = 12700'^{\#}$

$A_s = 12700 \times 12/18000 \times .867 \times 14.5 = .68^{\square''}/ft.$
$v = 3840/12 \times .867 \times 14.5 = 25^{\#}/_{\square''}$

3. At $h = 7.5'$

$M = 80 \times \frac{\overline{7.5}^2}{2} \times \frac{7.5}{3} = 5630'^{\#}$

$A_s = 5630 \times 12/18000 \times .867 \times 14.5 = .30^{\square''}$

WAR DEPARTMENT

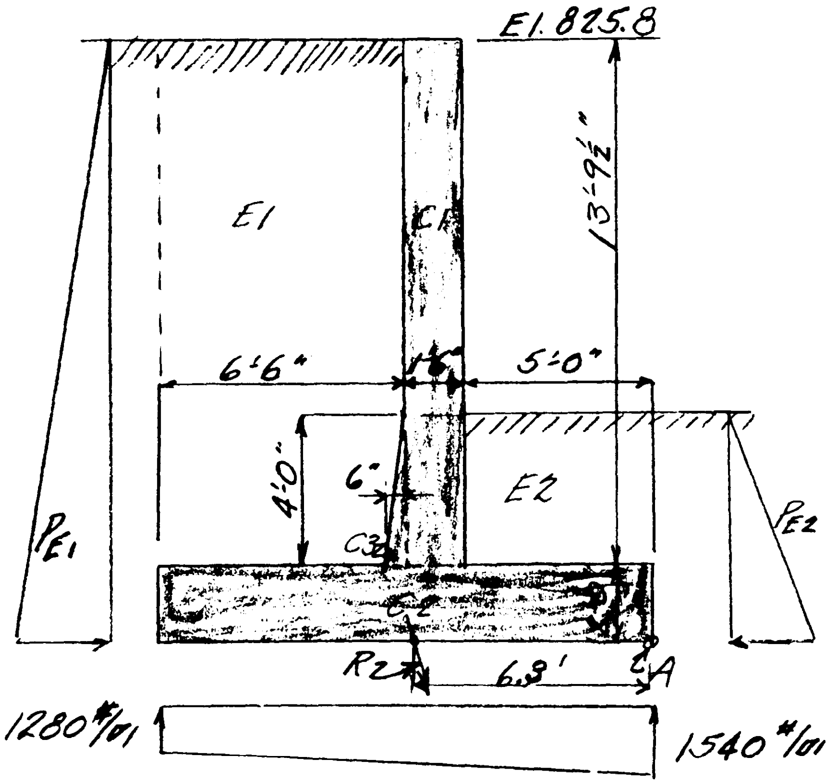
U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

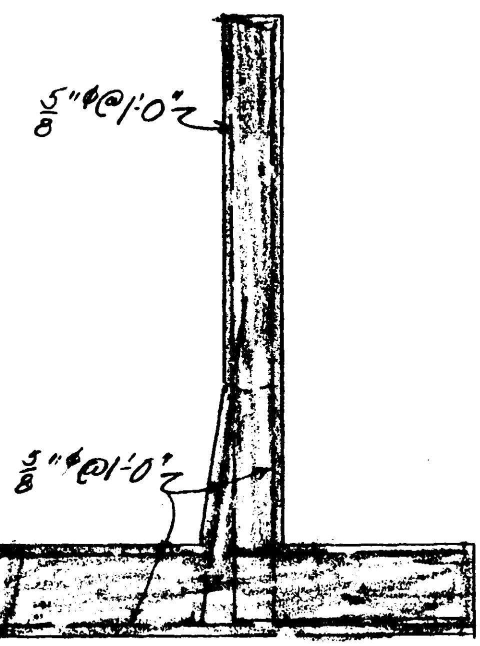
Subject: BIRCH HILL

Computation: Access road bridge

Computed by: RSM Checked by Date: 12-12-39

WING WALLS - DOWNSTREAM



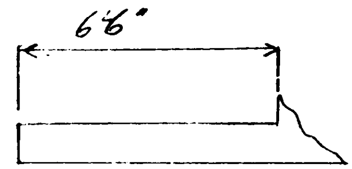


bject BIRCH HILL
omputation Access road bridge
omputed by RSM Checked by Date 12-12-39

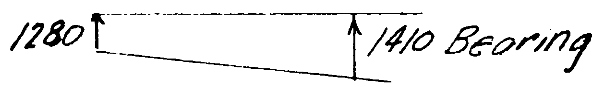
DESIGN OF BASE

1. Fill side

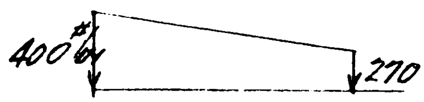
Loading $13.8 \times 100 = 1380$ #/□'
$2.0 \times 150 = \underline{300}$
1680 #/□'



	V		M
$270 \times 6.5 =$	$1750^{\#} \times 3.25 =$		$5700'^{\#}$
$130 \times 6.5/2 =$	$\underline{420} \times 4.33 =$		$\underline{1820}$
	$2170^{\#}$		$7520'^{\#}$



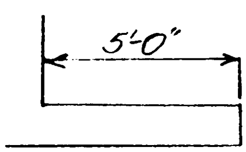
$$A_s = \frac{7520 \times 12}{18000 \times .867 \times 19.5} = .30^{\square''}$$



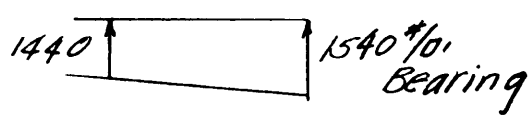
$$v = \frac{2170}{12 \times .867 \times 19.5} = 11 \ \#/\square'' \ OK$$

2. Stream side

Loading $4 \times 100 = 400$ #/□'
$2 \times 150 \quad \underline{300}$
700 #/□'

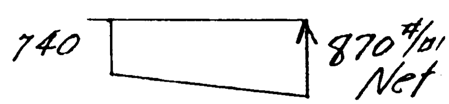


	V		M
$740 \times 5 =$	$3700^{\#} \times 2.5 =$		$9250'^{\#}$
$130 \times \frac{5}{2} =$	$\underline{325} \times 3.3 =$		$\underline{1070}$
	$4025^{\#}$		$10320'^{\#}$



$$A_s = \frac{10320 \times 12}{18000 \times .867 \times 19.5} = .41^{\square''}$$

$$v = \frac{4025}{12 \times .867 \times 19.5} = 20 \ \#/\square''$$



$$f_c = \frac{2M}{kjbd^2} = \frac{2 \times 10320 \times 12}{.409 \times .867 \times 12 \times 19.5^2} = 152 \ \#/\square''$$

bject BIRCH HILL
mputation Access road bridge
mputed by RSM Checked by Date 12-12-39

STABILITY

		Force ↓	Force ⇆	Arm	Moment ↷	abt. A ↶
E1	100×6.5×13.8	9000		9.75		88000
E2	100×5.0×4.0	2000		2.0		4000
C1	150×1.5×13.8	3100		5.75		17800
C2	150×2.0×13.0	3900		6.5		25300
C3	150×4.0×.5/2	300		6.7		2000
P_{E1}	$35 \times \overline{15.8}^2/2$		→ 4360	5.3	23100	
P_{E2}	$35 \times \overline{6.0}^2/2$		← 630	2.0		1260
		↓18,300			23100	138360
					ΣM =	115,260

Position of resultant $\frac{115260}{18300} = 6.3$, third pt. at 4.3'

$e = 6.5 - 6.3 = .2'$

$P = \frac{18300}{13}\left(1 \pm \frac{6 \times .2}{13}\right) =$ 1540 #/□' to 1280 #/□'

DESIGN OF STEM -

1. At base

	V			M
$\rightarrow 35 \times \frac{\overline{13.8}^2}{2} =$	3330 #	×	4.6	= 15300 '#
$\leftarrow 35 \times \frac{\overline{4}^2}{2} =$	280	×	1.33	370
	2050 #			14930 '#

$A_s = 14930 \times 12 / 18000 \times .867 \times 20.5 = .56$ □"

$v = 2050/12 \times .867 \times 20.5 = 10$ #/□".

2. At top of haunch

	V		M
$35 \times \overline{9.8}^2/2 =$	1680	× 3.3	5550 '#

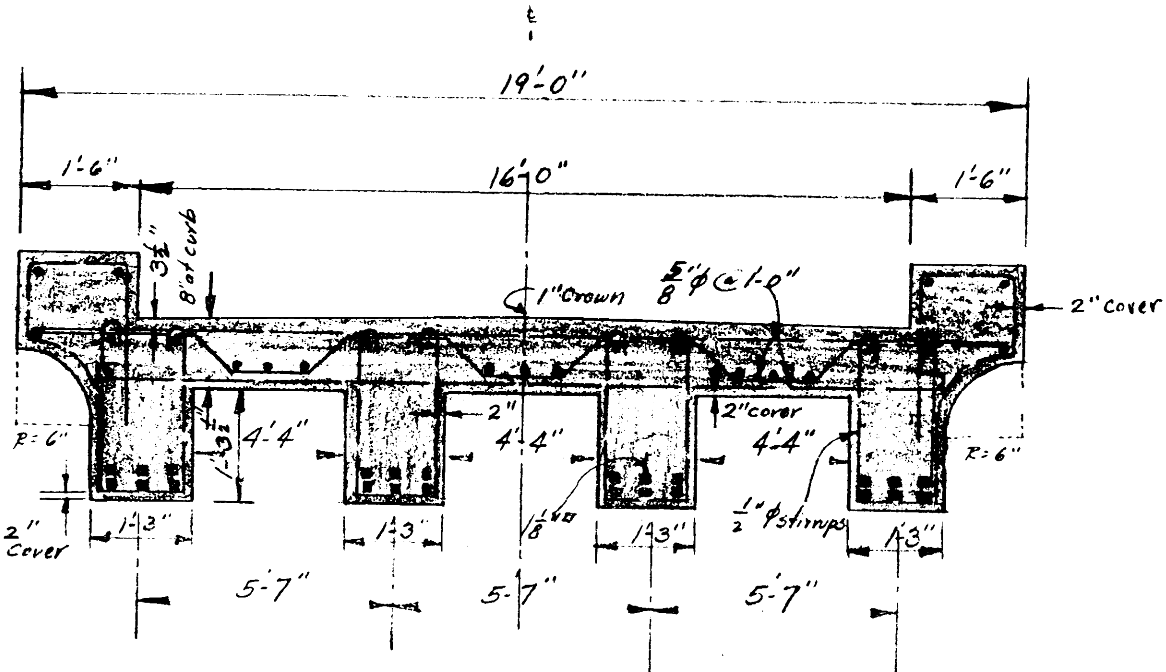
$A_s = 5550 \times 12/18000 \times .867 \times 14.5 = .30$ □"

$v = 1680/12 \times .867 \times 14.5 = 11$ #/□".

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Subject Birch Hill Dam
Computation Design of T-Beam Bridge with 24' clear span
Computed by J.A.C. **Checked by** RSM **Date** 12/14/39



Longitudinal steel: $\frac{5}{8}$"φ

f'_c = 2500 #/□"

f_s = 18000 #/□"

f_c = 1000 #/□"

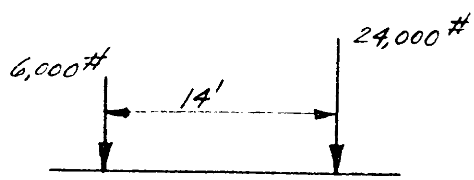
n = 12

24' clear span

25.5' effective span

Wearing course - 1½"

H-15 Loading



WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

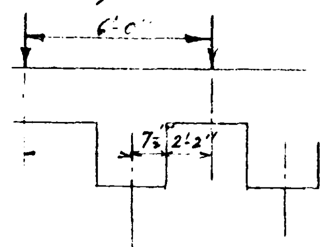
bject: Birch Hill
nputation: T-Beam Bridge - 24' clear span
mputed by: J.A.C. Checked by: RSM Date: 12/14/39

DESIGN OF FLOOR SLAB TRIAL DEPTH = $6\frac{1}{2}''$

Width of tread = 1"/Ton of Loaded Truck = $1 \times 15 = 15''$

Clear span = 4'-4"

E = Effective width of slab in feet for 1 wheel = $0.7(2 \times 2.79 + 1.25) = 4.8'$



Conc Load = $\frac{12,000}{4.8} = 2,500^{\#}$

$6'-0''$
$-2'-9\frac{1}{2}''$
$3'-2\frac{1}{2}''$
$D = 2'-9\frac{1}{2}'' = 2.79'$

MOMENT:

LIVE = $\frac{2500}{2} \times 2.16 \times 12 \times 0.8 = 25,900''^{\#}$

IMPACT = $\frac{50}{5.58+125} \times 25,900 = 9,600''^{\#}$

DEAD
$2\frac{1}{2}''$ Surfacing @ $30.0^{\#}/\square'$
Trial $6\frac{1}{2}''$ slab @ $81.5^{\#}/\square'$
$111.5^{\#}/\square'$

$D.M. = \frac{111.5 \times 4.33^2}{8} \times 12 \times 0.8 = 2,510''^{\#}$

TOTAL MOMENT = $38,010''^{\#}$

Balanced reinf. assumed

$d = \sqrt{\frac{M}{b \times R}} = \sqrt{\frac{38,010}{12 \times 174}} = \sqrt{18.2} = 4.25''$ $R = \frac{1}{2} f_c k j$

From table $R = 174.$
$k = .409$
$j = .867$

$A_s = \frac{M}{f_s j d} = \frac{38,010}{18,000 \times .867 \times 4.25} = 0.58^{\square''}$

$V = 2,500 + .383(2500) = 3460^{\#}$ (960)

$\boxed{\text{Use } \frac{5}{8}''\phi \text{ @ } 6'' \text{ c.c. } (A_s = 0.61^{\square''}) \text{ in bott. face, bending up alt. bars over the supports and additional } \frac{5}{8}''\phi \text{ bars are placed at } 12'' \text{ c.c. in the top face}}$

$v = \frac{3460}{12 \times .867 \times 4.25} = 78^{\#}/\square''$ OK.

$\Sigma_o = \frac{3460}{125 \times .867 \times 4.25} = 7.5$ Rqd. Bars must be hooked.

ction in the rock cut to ... ft./sec. at Station 17 + 00 where the chan-

Subject: Birch Hill

Computation: T-Beam Bridge - 24' Clear span

Computed by: J.A.C. Checked by: RSM Date: 12/14/39

Intermediate Beam Design

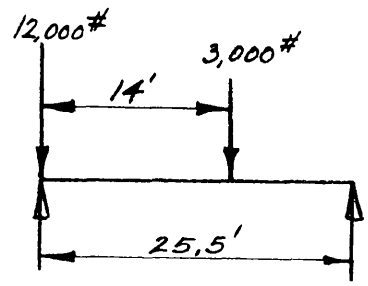
18" Abutments

25.5' Effective span - c. to c. of bearings

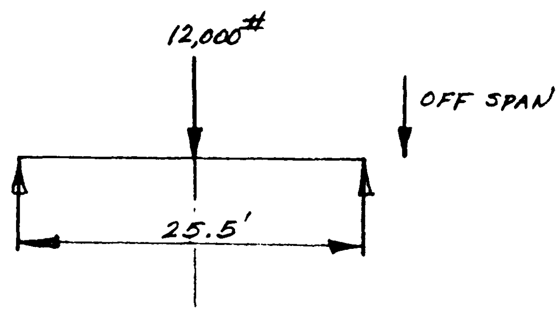
Live Load Shear and Moment

Positions of Loads for:

Max. Shear



Max Moment



$$\text{Max. L.L.V.} = 12{,}000 + \frac{11.5}{25.5}(3{,}000) = 13{,}350\#$$

$$\text{Max. L.L.M.} = \frac{12{,}000}{2} \times 12.75 = 76{,}500'\# = 918{,}000''\#$$

Dead Load Shear and Moment

Surface	$= 12.5 \#/\square'$
8" Slab	$= 100 \#/\square'$

$$112.5 \#/\square' \times 5.58 = 630 \#/\text{lin. ft.}$$

$$\frac{15.5 \times 15}{144} \times 150 = 240 \#/\text{lin ft}$$

$$\text{Total D.L./lin. ft} = 870\#$$

$$\text{D.L.V.} = \frac{1}{2} \times 870 \times 25.5 = 11{,}100\#$$

$$\text{D.L.M} = \frac{wl^2}{8} = \frac{870 \times \overline{25.5}^2}{8} \times 12 = 850{,}000 ''\#$$

en made to prevent erosion downstream or upstream from the crests of

Subject: BIRCH HILL

Computation: T-BEAM BRIDGE – 24' CLEAR SPAN

Computed by: J.A.C. Checked by: RSM Date: 12/14/39

INTERMEDIATE BEAM DESIGN

IMPACT SHEAR AND MOMENT

$$\text{Impact Factor} = \frac{50}{25.5+125} = 33\tfrac{1}{3}\%$$

I.V. = $33\frac{1}{3}\%$ of 13,350# = 4,450#

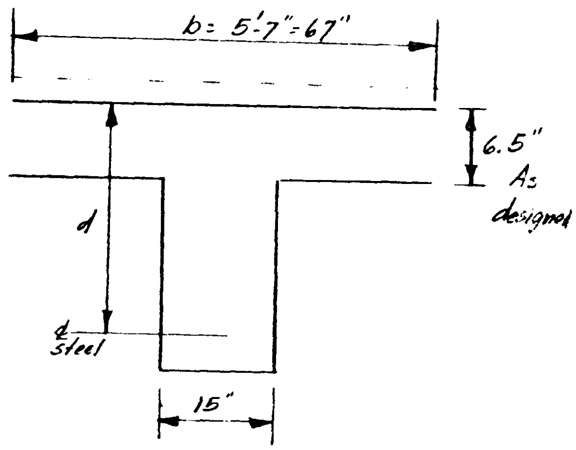
I.M. = $33\frac{1}{3}\%$ of 918,000"# = 306,000"#

SUMMARY OF SHEARS AND MOMENTS

	SHEAR	MOMENT
LIVE LOAD	13,350#	918,000"#
IMPACT	4,450	306,000
DEAD LOAD	11,100	850,000
TOTAL	28,900#	2,074,000"#

Subject: BIRCH HILL
Computation: T-BEAM BRIDGE - 24' Clear span
Computed by: J.A.C. Checked by: RSM Date: 12/14/39

INTERMEDIATE BEAM DESIGN



TRIAL #1

Assume $d = 18"$
$b = 67"$
$b' = 15"$
$t = 6.5"$
Assume 4" cover to ℄ steel making total depth = 22"
$22 - 6.5 = 15.5"$ Stem

$$k = \frac{12{,}000}{18{,}000 + 12{,}000} = 0.400 \qquad \frac{t}{d} = \frac{6.5}{18} = 0.361$$

$$jd = 18 - \frac{3(.4) - 2(.361)}{2(.4) - .361} \times \frac{6.5}{3} = 18 - 2.36 = 15.64"$$

$$C = \frac{M}{jd} = \frac{2{,}074{,}000}{15.64} = 132{,}600^{\#}$$

$$f_c = \frac{132{,}600}{\frac{.8 - .361}{.8} \times 67 \times 6.5} = \frac{132{,}600}{239} = 560 \ \#/in^2$$

$$A_s = \frac{132{,}600}{18{,}000} = 7.35 \ \square"$$

$$\boxed{\text{Use } 6 - 1\tfrac{1}{8}"\square \quad A_s = 7.59 \ \square" \quad \Sigma_0 = 27.0}$$

$$p = \frac{7.59}{67 \times 18} = .0063$$

$$k = \frac{.0063(12) + \frac{1}{2}(.361)^2}{.0063(12) + .361} = \frac{.0756 + .0651}{.4366} = .322$$

$$jd = 18 - \frac{3(.322) - 2(.361)}{2(.322) - .361} \times \frac{6.5}{3} = 18 - 1.86 = 16.14"$$

$$C = \frac{2{,}074{,}000}{16.14} = 128{,}500^{\#}$$

bject Birch Hill
nputation T-Beam Bridge – 24' Clear Span
mputed by J.A.C. Checked by RSM Date 12/15/39

Intermediate Beam Design

$$f_c = \frac{128,500}{\frac{.644-.361}{.644}\times 67\times 6.5} = \frac{128,500}{191.5} = 670\ \#/in^2$$ OK Allowable = 1,000

$$\frac{f_s}{f_c} = 12\left(\frac{1-.322}{.322}\right) = 25.3$$

$$f_s = 25.3\times 670 = 17,000\ \#/in^2$$ OK Allowable = 18,000

Investigation for Shear and Bond Stresses

$$v = \frac{V}{b'jd} = \frac{28,900}{15\times 16.14} = 120\ \#/\square''$$ Use Web Reinforcement

$$u = \frac{V}{\Sigma_o jd} = \frac{28,900}{27.0\times 16.14} = 66\ \#/\square''$$ OK for bond

Let 3 bars continue over support for bond
Bend 3" up for diag. tension.

Web Reinforcement

Stirrups

Unit Shear at 3'-3" = 49 #/□"

$$s = \frac{2\times .25\times 16,000}{49\times 15} = \underline{11''}\ \text{Use}$$

$$s = \frac{3}{4}\times d = \frac{3}{4}\times 18 = 13\tfrac{1}{2}''$$

Unit Shear at 6'-0"

$$s = \frac{2\times .25\times 16,000}{30\times 15} = 18''$$

$$s = \frac{3}{4}\times 18 = \underline{13\tfrac{1}{2}''}\ \text{Use}$$

Shears at:

End = Tot. V. = 28,900 #
Mid Span = L.L.V. + I.V. = 6,000 + 2200 = 8,200 #
← For 12,000 # Load at ℄.

$$v = \frac{V}{b'jd} = \frac{V}{15\times 16.14} = \frac{V}{242}$$

$$v\ (\text{At support}) = \frac{28,900}{242} = 120\ \#/\square''$$

$$v\ (\text{At mid span}) = \frac{8,200}{242} = 34\ \#/\square''$$

Bent up Bars

Arbitrary (1) $\text{Max. } s = \frac{45}{10+45}\times 18 = \underline{15''}$ Use

Calc. 2 Bars up at Support (2) $\text{Max. } s = \frac{2\times 1.265\times 16,000\times 1.414}{70\times 15} = 55''$

1 Bar up (3) $\text{Max } s = \frac{1\times 1.265\times 16,000\times 1.414}{57\times 15} = 33''$

he toes will be constructed to complete the full embankment section.

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

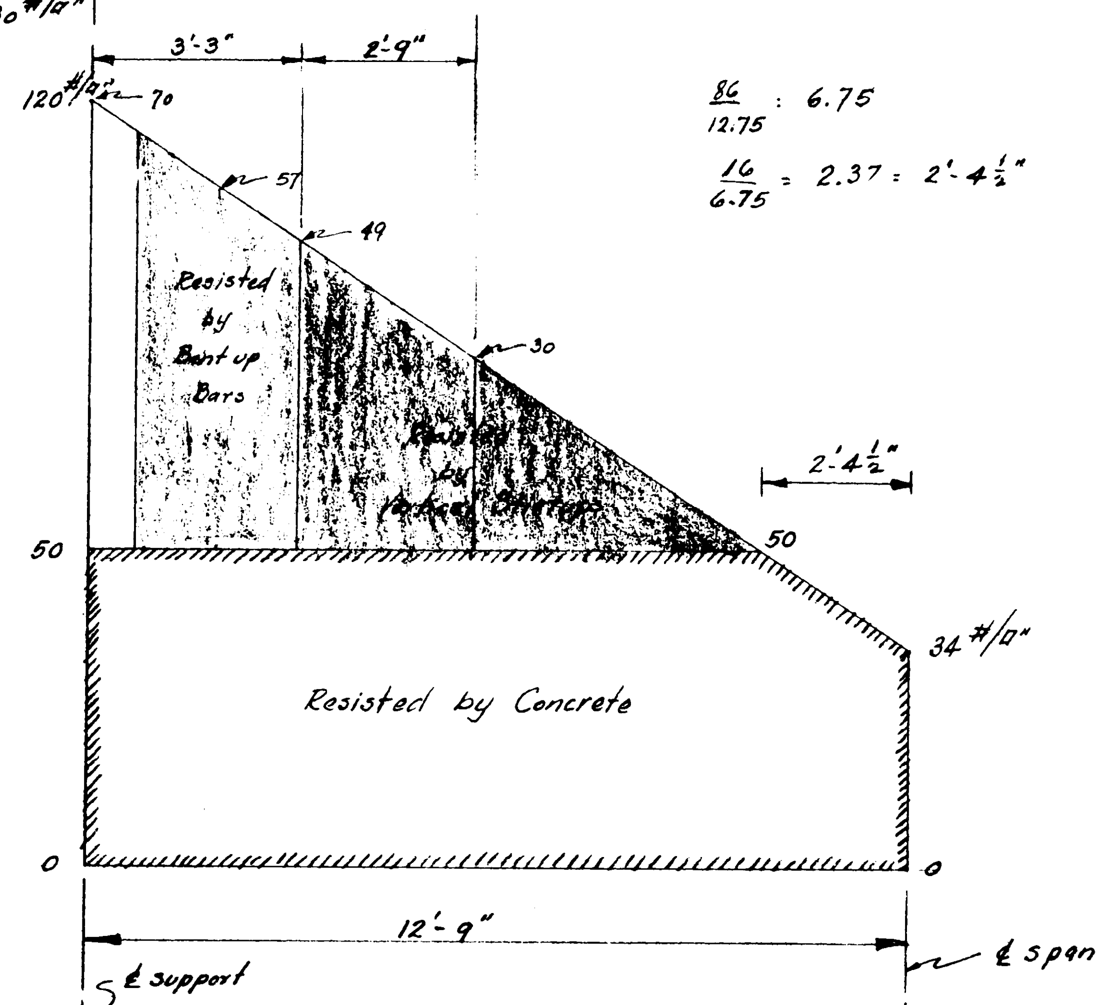
bject: Birch Hill

nputation: T-Beam Bridge - 24' Clear Span

nputed by: J.A.C. Checked by: RSM Date: 12/15/39

ale - 1" = 3'
rt - 1" = 30 #/□"

UNIT SHEAR DIAGRAM (TO SCALE)



$\frac{86}{12.75} : 6.75$

$\frac{16}{6.75} = 2.37 = 2'-4\frac{1}{2}"$

120
− 34
86

3-3
2-4½
5-7½

12-9
5-7½
7'-1½"

-1" = 3'
-1" = 3'